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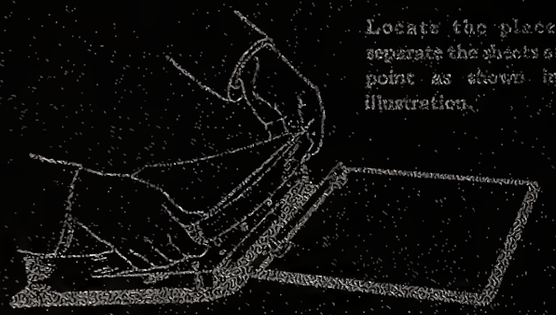
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SECTION IV

WAVE FORCES ON OBJECTS, PILING,
VERTICAL WALLS, AND RUBBLE MOUNDS

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# MANUAL OF AMPHIBIOUS OCEANOGRAPHY

## SECTION IV: WAVE FORCES ON OBJECTS, PILING, VERTICAL WALLS AND RUBBLE MOUNDS

BY
J. R. MORISON

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# SUMMARY OF SECTION IV

## WAVE FORCES ON STRUCTURES

BY
J. R. MORISON

Ocean waves exert forces on man-made structures and natural objects which tend to move or destroy them. As a wave encounters an object the force acts first in the direction of wave travel, with the crest, and then in the opposite direction, with the trough. This reversal of force which occurs is similar to that which causes a pebble to be rolled up and down a beach by the uprush and backwash of waves.

There are several distinct kinds of wave forces. Waves tend to drag and push a structure or object, which causes movement and erosion. Waves may break against a structure and thus tend to break it. Figure SIV-1 shows a wave breaking against a grounded ship which was eventually broken in half. Waves may rise up a wall or breakwater, exerting a pressure that may overturn the wall.

Wave forces on objects such as spheres and cylinders can be computed using the equations of wave particle motion for unbroken waves. The forces and moments on structures such as piles and walls can be computed similarly. Detailed explanations of these computations are presented in Section IV on "Wave Forces on Objects, Piling, Vertical Walls, and Rubble Mounds" of this Manual.

Many examples of the effects of wave forces have been observed by most persons who have been exposed to the sea; many persons have had the experience of being rolled by breakers in the surf zone. Freely floating objects on the surface of waves move back and forth and slowly propagate in the direction of the wave travel. Ships roll when broadside to short period waves and often surge slowly when tied to a dock, under the action of "long period" oscillations. Figure SIV-2 shows an example of damage to a pier by the surging of a ship. All these motions are obvious results of the force of waves and demonstrate the reversal of the direction of wave force as the crest and trough pass.

The force of waves which break spectacularly against walls or cliffs, where spray is shot upward, is difficult to explain. In this case, air which is often trapped by the breaking waves and compressed against the cliff causes the explosive effect of the spray and the crashing sound, the degrees of which depends upon the amount of entrapped air. The less the air the shorter the explosive sound and the higher the force. It is best not to build walls or structures where the ocean waves can break directly on them.

Waves damage piers and platforms which are too close to the water surface, by pounding the decking off from underneath as the water rises with the wave crest. Roads, breakwaters, seawalls, and similar structures are damaged by spray and water from the waves falling onto the structures. Figure SIV-3 shows water from waves pouring on a breakwater. Shore line structures such as seawalls and bulkheads are often undermined and robbed of the backfill when water from the waves is not drained off the structures properly. Figure SIV-5 shows a wall of a hotel far above the water which has lost its foundation due to the erosion and undermining of the shore lines. For more examples, of wave damage and wave force, see Section IX, "Invasion Harbors" of this Manual.

To demonstrate how the forces of waves vary with the wave height, the wave period and the water depth, Figure SIV-5 was drawn: it shows the maximum total moment exerted by waves about the bottom of a pile. The moment increases with increasing wave height, wave period, and water depth (except in shallow water where the moment is independent of the wave period because the wave velocity is approximately a constant for a given depth).

Table SIV-I shows the relationship of the waves to the maximum total moment and the phase angle of the examples plotted in Figure SIV-5. Notice that the maximum total moment occurs ahead of the wave crest for large wave heights, short wave periods, and relatively deep water. Also, this occurs for larger pile diameters.

The total moment is given by the expression

$$M_{max} = \rho_w \frac{D\, H^2\, L^2}{T^2} \left[ C_d\, k_3\, \mathrm{Cos}\beta^2 + \frac{\pi D}{4 H}\, C_M k_4\, \mathrm{Sin}\beta \right] \qquad \text{Foot/pounds}$$

The angular position of maximum moment on the pile before the crest reached the pile is given by the expression

$$\mathrm{Sin}\beta = \frac{\pi D\, C_M}{8 H C_d} \frac{k_4}{k_3}$$

where

$\rho_w$ = density of water = 2.0 slugs/ft$^3$
D = pile diameter = 1 foot
H = wave height = 1 to 20 feet
L = wave length = 83 to 510 feet
T = wave period = 5, 10, and 15 seconds
$C_d$ = coefficient of drag = 1.63
$C_M$ = coefficient of mass = 1.51
d = still water depth = 10, 20, 30, and 40 feet

$$k_3 = \frac{\frac{1}{2}\left(\frac{4\pi d}{L}\right)^2 + \frac{4\pi d}{L}\,\mathrm{Sinh}\,\frac{4\pi d}{L} - \mathrm{Cosh}\,\frac{4\pi d}{L} + 1}{64\left(\mathrm{Sinh}\,\frac{2\pi d}{L}\right)^2}$$

$$k_4 = \frac{\frac{2\pi d}{L}\,\mathrm{Sinh}\,\frac{2\pi d}{L} - \mathrm{Cosh}\,\frac{2\pi d}{L} + 1}{2\ \mathrm{Sinh}\,\frac{2\pi d}{L}}$$

Figure SIV-6 shows the meaning of some of the symbols. The expression for the maximum total moment has been simplified for this example and is therefore only approximately correct. A further simplification was made in that the coefficients of drag and mass were assumed constants for all the wave conditions. This assumption was made because the exact variation of the coefficients with different wave conditions is not known. The

values of the coefficients used were obtained from a series of model studies. For details see Section IV of this Manual.

The calculations or estimations of wave forces, especially those in the breaker zone, are an important part of any amphibious operation. A great deal of information is available on wave force and a great deal more information is needed to complete the analysis of wave force. No matter how complicated the problem of wave forces on structures and objects may seem, it is essential to any operation that it be investigated to the fullest extent.

TABLE S IV-I

MOMENT EXERTED ABOUT BOTTOM OF PILING FROM WAVE ACTION, FOOT-POUNDS

| WAVE PERIOD T | | 5 | 5 | 5 | 5 | 10 | 10 | 10 | 10 | 15 | 15 | 15 | 15 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| WAVE DEPTH d | | 10 | 20 | 30 | 40 | 10 | 20 | 30 | 40 | 10 | 20 | 30 | 40 |
| WAVE LENGTH L | | 83 | 106 | 118 | 123 | 174 | 244 | 291 | 329 | 266 | 378 | 455 | 517 |
| WAVE HEIGHT H | 2 | 32<br>346 | 50<br>722 | 69<br>1,246 | 90<br>1,813 | 15<br>269 | 21<br>582 | 27<br>896 | 32<br>1,233 | 10<br>266 | 14<br>599 | 17<br>842 | 20<br>1,128 |
| | 4 | 15<br>1,133 | 22<br>2,079 | 28<br>3,235 | 32<br>4,413 | 8<br>1,026 | 10<br>2,124 | 13<br>3,132 | 15<br>4,153 | 5<br>1,101 | 7<br>2,144 | 8<br>3,156 | 10<br>4,151 |
| | 6 | 10<br>2,458 | 15<br>4,375 | 18<br>6,553 | 20<br>8,749 | 5<br>2,285 | 7<br>4,788 | 9<br>6,865 | 10<br>8,993 | 3<br>2,339 | 5<br>4,788 | 6<br>7,029 | 7<br>9,178 |
| | 8 | | 11<br>7,576 | 13<br>11,200 | 15<br>14,810 | | 5<br>8,314 | 6<br>12,086 | 8<br>15,781 | | 3<br>8,474 | 4<br>12,436 | 5<br>16,239 |
| | 10 | | 9<br>11,676 | 11<br>17,555 | 12<br>22,621 | | 4<br>12,919 | 5<br>18,784 | 6<br>24,506 | | 3<br>13,247 | 3<br>19,377 | 4<br>25,326 |
| | 12 | | 7<br>16,710 | 9<br>24,494 | 10<br>32,173 | | 3<br>18,587 | 4<br>26,949 | 5<br>35,164 | | 2<br>19,039 | 3<br>27,904 | 3<br>36,368 |
| | 14 | | 6<br>22,639 | 8<br>33,099 | 9<br>43,436 | | 3<br>25,299 | 4<br>36,669 | 4<br>47,777 | | 2<br>25,889 | 2<br>37,943 | 3<br>49,501 |
| | 16 | | | 7<br>43,060 | 8<br>56,360 | | | 3<br>47,822 | 4<br>62,402 | | 2<br>33,880 | 2<br>49,512 | 2<br>64,593 |
| | 18 | | | | | | | 3<br>60,580 | 3<br>78,838 | | 2<br>42,839 | 2<br>62,604 | 2<br>81,675 |
| | 20 | | KEY<br>β<br>M | | | | | 3<br>74,654 | 3<br>97,327 | | 1<br>52,886 | 2<br>77,435 | 2<br>100,832 |

$\beta$ = Phase angle, degrees

M = Total moment about bottom of pile, Foot-pounds

D = Pile diameter = 1 foot



FIG. SIV-1 - The grounded *Ioannis J. Kulukundis.* Waves would break against the ship at low tide. (7-11-49)





FIG. SIV-3 - Water from waves passing over the Morro Rock breakwater.

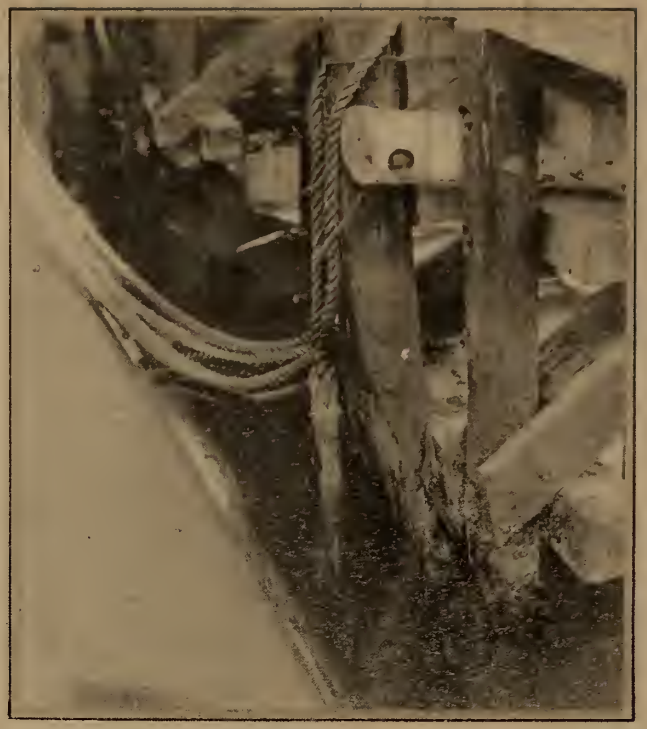


FIG. SIV-2 - Damage to piling caused by the surging motion of the submarine *Carman.* (7-8-46)



FIG. SIV-4 - Damaged wall of hotel at Kincheloe Point, Oregon. (9-18-45)

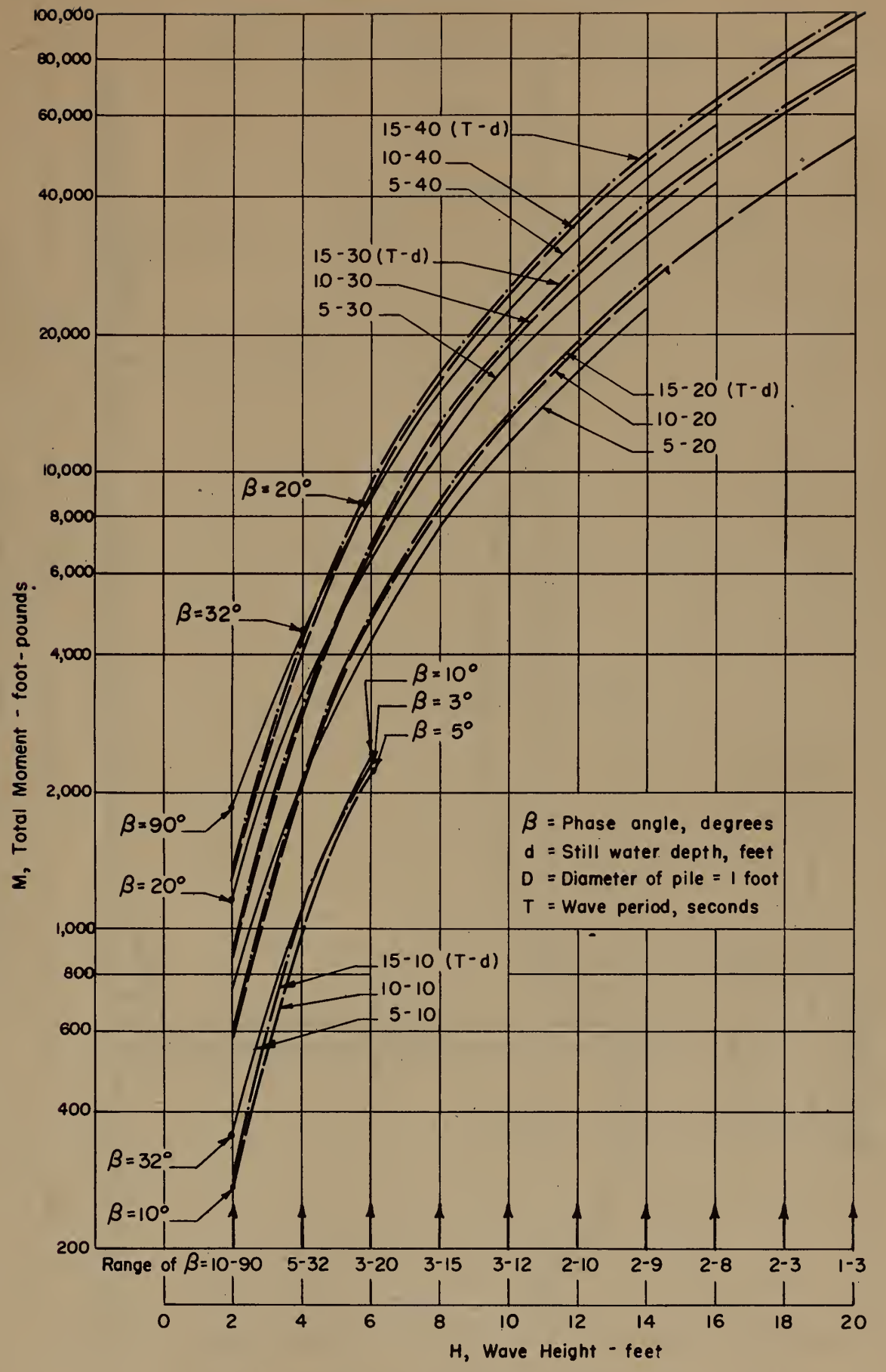



FIG. SIV-5 - TOTAL MOMENT ABOUT BOTTOM OF PILING FROM WAVE ACTION

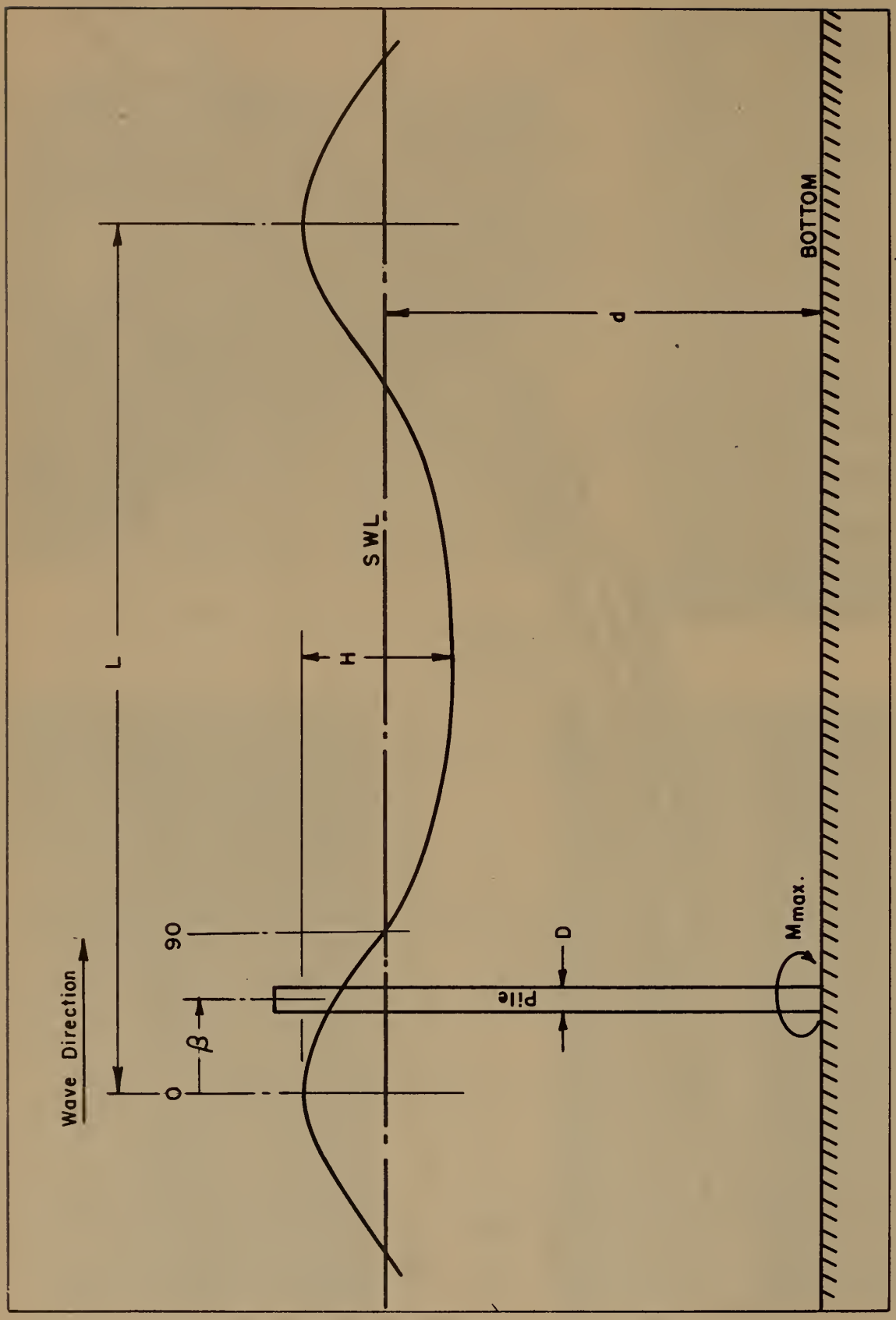



FIG. SIV-6 - WAVE SYMBOL DEFINITION

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SECTION IV: WAVE FORCES ON OBJECTS,
PILING, VERTICAL WALLS AND RUBBLE MOUNDS

## A: ABSTRACT

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# SECTION IV

## WAVE FORCES ON OBJECTS, PILING, VERTICAL WALLS AND RUBBLE MOUNDS

### A. ABSTRACT

At present the wave force theory and experimental verification is limited to moderately shallow and deep water ($d/L > 0.200$) conditions. The theory for the forces exerted by surface waves on objects, piles, vertical walls, both stationary and floating and rubble mounds is presented. Experimental verification is presented for the objects, piles, vertical walls (stationary) and rubble mounds. The components of wave force studies are the drag force, the inertia force, the "hydrostatic" force and the impact force. The drag force and inertia force are the components present when small objects and piles are subject to small unbroken waves. The "hydrostatic" force on a vertical wall is the force resulting from fully reflected unbroken waves. Impact forces result from breaking waves. The rubble mound problem is studied from the standpoint of the stone size and density that will resist wave action and the slope that is stable for the rubble. The primary purpose of this section is to present a summary of the available information on wave forces and to point out that additional model studies and prototype experiments are needed for checking the theory.

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# SECTION IV: WAVE FORCES ON OBJECTS, PILING, VERTICAL WALLS AND RUBBLE MOUNDS

## B: INTRODUCTION

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## B. INTRODUCTION

Structures which are placed in the ocean are subject to the forces exerted by waves. This type of repeated force is the importance in the design of structures which are to have a long period of usefulness. A knowledge of the wave force is especially important if the structure is to withstand storm waves. From the cost viewpoint, the over-designed structure is as unsound as the under-designed structure. The military view is somewhat different in that the time of construction is the important factor and over-strength structures take valuable time to build. Thus in both cases, every factor considered in design must be known accurately. Although the sea is somewhat unpredictable in its actions, the forces resulting from known action of the sea can be computed accurately enough so that structures built in the sea will withstand the wave action for which they were designed. There are many examples of structures exposed to the sea that have withstood storm waves and many other examples of under-strength structure that have not withstood the sea.

The components of wave forces are drag (friction) forces, inertia forces, "hydrostatic" forces and impact forces. When a wave breaks against a wall all these components of forces are experienced with the predominate force initially being impact, followed by the "hydrostatic" force of the water piled upon the wall from the wave. The drag and inertia forces are masked by the other forces so that they are not distinguishable although their actual magnitude may be a relatively large (indicated by the pressure recorded when the trough acts on the wall) part of the total forces acting on a wall. The force of small waves on piling is composed of the drag force and the inertia force both of the same order of magnitude. Any "hydrostatic" load due to the shape of small waves is included in the expression for the inertia. If the waves are large and steep an additional "hydrostatic" load and an impact load may be experienced due to the water at the crest of the wave running up the piling or striking the pile abruptly as the crest passes the pile causing spray to be thrown into the air. When a standing wave pattern causes pressure on a wall, the predominate force is "hydrostatic". The so-called "hydrostatic" force of waves on a wall is actually reduced in magnitude due to the vertical velocity of the water along the wall and hence is not a purely hydrostatic force. The force of a wave breaking on a rubble mound is a combination of all the types of forces and is further complicated by the compression of trapped air. The severity of breaking and the amount of reflection of a wave from a sloping surface depends upon the wave period, the wave height, and the slope of the surface. It becomes relatively easy to compute the forces where the action of the waves in the vicinity of walls, piles, and objects can be described accurately by mathematical expressions. However, where the action has not been mathematically described, empirical data gathered from observations and measurements on model and prototype structures must be used.

It would take many pages to describe all the theories developed and observations taken during the last 250 years during which the modern wave theory has been developed. Reproduced here are only the well-known and widely used theories and empirical equations of the past years and the most recent and up-to-date extension of the modern wave theory as pertaining to the forces exerted by waves. As in most fields of engineering there are many problems that have not been solved. Examples of important unsolved problems are the forces exerted by breaking waves and by waves in water that is extremely shallow as compared to the wave length. Measurements of the exact amplitude of the forces of waves that break against a vertical wall are needed for comparison with the theoretically determined pressures. The wave forces that move sand should be studied both experimentally and theoretically so that the speculation on how sand is deposited can be eliminated or at least reduced to a study of local wave and current conditions.

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## C. FORCES ON OBJECTS

### 1. Wave Force on a Small Object Fixed Below the Water Surface (Reference IVC-10)

The theory of the wave forces exerted on a fixed object, although not developed first historically, is the logical beginning point of a wave force study. In a frictionless, incompressible fluid, the dynamical equation (Reference IV C-5) describing the motion of a small element of the liquid is

$$dxdydz\left(X - \frac{1}{\rho_w}\frac{\partial p}{\partial x}\right) = \left(\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} + v\frac{\partial u}{\partial y} + w\frac{\partial u}{\partial z}\right) dxdydz \qquad \text{(IV C-1.1)}$$

Here, X is the component of an extraneous force such as gravity, being zero if the x-axis is horizontal in a gravitational field. Oscillatory surface waves are limited in their height-length ratio and hence is approximately correct (Reference IV C-5) to assume that the convective terms $u\,\partial u/\partial x$, $v\,\partial u/\partial y$, and $w\,\partial u/\partial z$ are negligible as compared with $\partial u/\partial t$. The equation of motion becomes

$$-\frac{\partial p}{\partial x}\,dxdydz = \rho_w \frac{\partial u}{\partial t}\,dxdydz \qquad \text{(IV C-1.2)}$$

where

p = pressure

$\rho_w$ = density of water

x = horizontal coordinate

y = vertical coordinate

z = breadth coordinate

u = particle velocity in x - direction

v = particle velocity in y - direction

w = particle velocity in z - direction

t = time

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If the left-hand term is integrated throughout a selected volume, the integral yields the component of the normal pressure in the x-direction exerted on the liquid volume and this must, of course, equal the integrated acceleration of the liquid mass within the selected volume, but this macroscopic result does not yield information regarding the acceleration of the individual fluid masses within this volume. If the volume chosen is filled with a solid object, the acceleration of this object can be predicted from the pressure distribution provided that the presence of the object does not materially alter the flow, and this condition requires that the object replace a volume of liquid within which the velocities and acceleration are substantially uniform. In terms of objects exposed to oscillatory wave velocities, the object should be small in comparison with the wave length. Assuming this condition to be met, the integrated form of equation (IV C-1.2) becomes

$$F_p = M_o \frac{\partial u}{\partial t} \qquad \text{(IV C-1.3)}$$

where $M_o$ = total mass of body

When a force is applied to an object immersed in a fluid, acceleration will occur. The fluid surrounding the object acquires a velocity and the flow field gains kinetic energy. The force required to produce a given acceleration is greater than that necessary to produce the same acceleration of the same object in vacuo because work must be done on the flow field surrounding the object. This effect may be treated in various ways, the most usual being to regard the immersed object as having an added or "virtual mass", dependent upon the flow pattern around the object. The shape of the object is only one factor in determining the flow field and the virtual mass. The other factor is the state of the flow around the object, dependent primarily on the Reynolds number and on the degree of separation. Experiments on spheres in steady flow show a variation of the drag coefficient with both Reynolds number and the separation, that is the point of separation of the flow. It is to be expected that the virtual mass would exhibit similar variations; however, the point of separation and the virtual mass may not be the same in accelerated flow as in steady flow at the same relative velocity. Adding a term to represent the virtual mass, equation IVC-1.3 becomes

$$F_p = (M_o + M_v) \frac{\partial u}{\partial t} \qquad \text{(IV C-1.4)}$$

where $M_v$ = total virtual mass

The preceding discussion of the force exerted on submerged objects dealt with frictionless flow. In any real flow, there is an additional force, due to viscous friction, having a magnitude which depends upon the Reynolds number. At very low Reynolds numbers, the drag force will consist, predominantly, of the shearing forces over the surface of the object. When the turbulence of the approaching stream and the Reynolds number or shape effect combine to produce a turbulent boundary layer and separation, the friction force is determined primarily by the shape of the object and the force is usually represented by the equation

$$F_D = \tfrac{1}{2} C_d w A_p u^2/g \qquad \text{(IV C-1.5)}$$



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Here, A is the projected area perpendicular to u, the relative velocity. This equation may be employed to represent the drag force in viscous flow as well, where the drag force is proportional to the first power of u. However, it is not particularly convenient in this flow regime because the coefficient of drag is then an inverse function of u. This discussion is aimed primarily at conditions under which the flow is turbulent and $C_d$ (coefficient of drag) is substantially constant.

In order to combine equations IV C-1.4 and 1.5 to yield the total force on an immersed object the assumption must be made that all dimensions of the object are so small compared with the wave length that the object produces only a local and limited disturbance of the wave orbital velocities.

This analysis merely provides a basis for the correlation of data. Its validity will be tested by comparing the coefficients of virtual mass and drag obtained in oscillatory flow with those for the same objects under steady flow conditions. Exact agreement is not to be expected but the coefficients of drag and of virtual mass should be of the same order magnitude as in rectilinear flow. The total force exerted by wave action on an object is computed from the equation

$$F = \frac{1}{2}\rho_w C_d A_p u^2 + \rho_w C_M V_m \frac{\partial u}{\partial t} \qquad \text{(IV C-1.6)}$$

where $(M_o + M_v)$ in equation IV C-1.4 has been replaced by $\rho_w C_M V_m$ ($\rho_w$ is the water density, $C_M$ is a coefficient of inertia, and V is the volume of water displaced by the object). In the first part of equation IV C-1.6 $\rho_w$ has replaced $\frac{w}{g}$ of equation IV C-1.5. The first term of the right-hand side of equation IV C-1.6 is the drag force and the second term is the inertia force. If this expression is to give accurate results, the object size must be many times smaller than the wave length. The object size should be smaller than the wave height or, at most, very little larger. The wave length (L) is defined as the distance between successive wave crests (See Figure IV C-2). The wave height (H) is the vertical distance from the preceeding trough to the following crest. The object size is defined as the maximum width of the object's projected area. The projected area is the area of the object projected on a plane perpendicular to the direction of the particle velocity as shown in Figure IV C-1.

The wave theory used herein for computing wave forces is that presented by Sir G. G. Stokes (Reference IV C-13). The particle velocity in the horizontal direction is given by the equation

$$u = \frac{\pi H}{T} \frac{\text{Cosh } \frac{2\pi S}{L}}{\text{Sinh } \frac{2\pi d}{L}} \text{Cos } \Theta \qquad \text{(IV C-1.7)}$$

The particle velocity in the vertical direction is given by the equation

$$v = + \frac{\pi H}{T} \frac{\text{Sinh } \frac{2\pi S}{L}}{\text{Sinh } \frac{2\pi d}{L}} \text{Sin } \Theta \qquad \text{(IV C-1.8)}$$

where : $S = d - y + \eta$
$d$ = still water depth
$y$ = depth below the still water depth to the mean particle position
$\eta$ = vertical particle displacement as referred to the mean particle position

T = wave period, that is, the time taken for two successive wave crests to pass a known point.

$\theta$ = angular position of water particle in its orbit (degrees). The particle during a passing wave cycle moves through decreasing angle. Further, the angles $\theta = 2\pi(x-ct)/L$ as used this theory implies that the horizontal particle velocity is zero at the mean elevation of the particle. This simplifying assumption is very good for the wave condition as outlined in the theory.

Figure IV C-2 shows the physical meaning of the above symbols. The particle accelerations corresponding to the u and v velocities are

$$\frac{\partial u}{\partial t} = + \frac{2\pi^2 H}{T^2} \frac{\text{Cosh } \frac{2\pi S}{L}}{\text{Sinh } \frac{2\pi d}{L}} \text{Sin } \theta \qquad \text{(IV C-1.9)}$$

$$\frac{\partial v}{\partial t} = - \frac{2\pi^2 H}{T^2} \frac{\text{Sinh } \frac{2\pi S}{L}}{\text{Sinh } \frac{2\pi d}{L}} \text{Cos } \theta \qquad \text{(IV C-1.10)}$$

where

$\frac{\partial u}{\partial t}$ = horizontal particle acceleration in feet/(second)$^2$

$\frac{\partial v}{\partial t}$ = vertical particle acceleration in feet/(second)$^2$

These equations of particle velocity and acceleration describe, within about 10% the natural particle motion of a steady train of uniform waves in all depths of water where d/L is greater than 0.200. They can be used to get approximate solutions to depth where d/L is as low as 0.100, but caution must be used.

Since in the equation for S, the term $\eta$ is a function of y as well as d, H, and L, the equation of force is quite complicated. (S is actually the distance from the bottom to the water particle.) The particle velocity and acceleration change over the length of the object. However, if the object is small and is fixed below the surface of the water, then the velocity and acceleration of the centerline of the object can be used. That is by using, y equal to the depth to the centerline of the object and $\eta$ equal to zero, a good approximation can be obtained for the velocity and acceleration distribution which is now assumed to be nearly constant over the height of the object. This generally means that the object, if larger than a wave height, must be situated near the bottom for the above relationships to apply. If the object is small compared to the wave height, it can be at any elevation as long as it is below the trough. For large objects and objects lying between the trough and the crest, the effect of the velocity distribution must be computed. Figure IV C-3 shows these approximations and the types of velocity distributions that are possible.

For any given location of an object and any known ocean wave acting on the object, the following quantities and wave characteristic must be known:

(i) Wave height (H)
(ii) Wave length (L)
(iii) Wave period (T)
(iv) Still-water depth (d)

(v) Projected area of object ($A_p$)
(vi) Volume of object ($V_m$)
(vii) Density of water (generally taken as 2.0 slugs/ft$^3$) ($\rho_w$)
(viii) Elevation of the centerline of the object above the bottom (S)

The wave height, the wave period and the still-water elevation at a particular location are obtained by direct observation or from the historical records of wave action at the site. Observed wave length should be used if possible; otherwise they can be calculated, knowing the wave period and water depth. The design wave should represent the wave conditions that exist at the site and should be a large enough wave so that no appreciable damage will occur during the expected life time of the structure. The frequency and severity of waves, if any, larger than the design wave should be known and the damage resulting from these waves should be calculated. It is rarely possible to design for the maximum expected wave during the life time of a structure, hence careful consideration of the design wave is necessary. If all the above listed quantities are known, the force on the object can be computed provided the size of the object does not greatly effect the flow pattern. Another important factor is that equation IV C-1.6 for the force is based on a constant depth so that the bottom conditions must be known. If the object is on a sloping beach then the theory applies only in water deep enough so that the bottom can be approximated by a constant depth. This depth is expressed as a ratio of the wave length and the still-water level. If $d/L$ is greater than 0.20 ($d/L_o = 0.17$), then equation IV C-1.6 for the force may be used provided the bottom is not irregular; that is, having high ridges and low valleys.

In equation IV C-1.6, the coefficient of drag ($C_d$) and the coefficient of mass ($C_M$) must be determined, either experimentally or analytically, before computation of the wave force can be made. For an object fixed below the water surface, where a constant velocity distribution is assumed, the coefficients pertaining to these conditions should be used. The coefficient of drag in this case can be obtained from Table IV C-1 and Figure IV C-4 (Reference IV C-6, 7, 9 and 10) in the following manner. First, compute $R_e$ using the characteristic length (D) (the maximum width of the projected area), the maximum centerline particle velocity as given by equation IV C-1.7 and 1.8 and the kinematic viscosity. The coefficients of drag presented in Table IV C-1 and Figure IV C-4 have been determined experimentally. The Reynolds number is given by the equation

$$Re = \frac{Du}{\nu} \qquad \text{(IV C-1.11)}$$

The symbol, $\nu$, (nu) is the kinematic viscosity of the water taken as $1.084 \times 10^{-5}$ Ft$^2$/sec for fresh water and $10^{-5}$ Ft$^2$/sec for salt water. Many of the values are for steady flow conditions and must be used with caution. All values given for wave or oscillatory motion are from model studies and should be used with caution in prototype calculations. The coefficient of drag is assumed a constant over the height object and during one wave cycle.

The coefficient of mass ($C_M$) is determined from the object size, shape, and the wave or flow characteristic. Very little information is available concerning the coefficient of mass which is referred to as the virtual mass, or the mass of the water displaced plus the added, or apparent, or virtual mass. Values of $C_M$ have been determined from experiments by model wave studies, pendulum type experiment, and by forced oscillatory systems. The values are for model studies and should be used with caution in prototype

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calculations. Table IV C-2 gives values of $C_M$ for various Reynolds numbers, although no relationship between $C_M$ & Re seems to exist. Where several values are given in Table IV C-2, they were obtained from several sources. The coefficient of mass increases as the size of the object decreases and increases as the period of oscillation becomes longer. These two phenomena oppose each other in that the relative size of the object is compared to the wave length. For the purpose of the calculation discussed herein the coefficient of mass is assumed to be constant throughout the height of the object and throughout the wave cycle.

The component parts of equation IV C-1.6 have been discussed with relation to a fixed object. The equation IV C-1.6 itself is periodic by definition of the terms $u^2$ and $\frac{\partial u}{\partial t}$. Therefore, the force has a maximum at some angular position ($\theta = \beta$) of the water particle. The angular position ($\beta$) of the maximum force is sometimes referred to as the phase angle. It may lead or lag the crest of the wave ($\theta = 0$). Substituting the equations IV C-1.7 and 1.9 into equation IV C-1.6, the horizontal force ($F_H$) on an object is obtained. (Had equation IV C-1.8 and 1.10 been used the vertical force would have been obtained.)

$$F_H = \pm \frac{\pi^2}{2} \rho_w C_d A_p \frac{H^2}{T^2} K^2 \cos^2 \theta + 2\pi^2 \rho_w C_M V_m \frac{H}{T^2} K \sin \theta \qquad \text{(IV C-1.12)}$$

where

$$K = \frac{\cosh \frac{2\pi S}{L}}{\sinh \frac{2\pi d}{L}} \qquad \text{(IV C-1.13)}$$

The plus and minus sign of the first term of equation IV C-1.12 are applied to the $\cos^2\theta$ and take on the plus or the minus as would the $\cos \theta$ during the particle angular travel as shown in Figure IV C-2. By differentiating equation IV C-1.12 with respect to $\theta$ and setting it equal to zero, the angular position of the maximum horizontal force on an object fixed below the water surface is obtained

$$\sin \beta = \frac{2 C_M V_m}{C_d A_p H K} \qquad \text{(IV C-1.14)}$$

where

$\beta$ = angular position of the water particle when the maximum horizontal force occurs. $\beta$ is a value of $\theta$.

From the equations IV C-1.12 and 1.14 or similar equations, the force and position of maximum force can be calculated. The force at any angular position (Reference IV C-13) can be calculated and the location of this position along the wave profile can be calculated from the expression

$$\frac{x}{L} = \frac{\theta}{360} - \frac{H}{2L} \frac{1}{\tanh \frac{2\pi d}{L}} \sin \theta \qquad \text{(IV C-1.15)}$$


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where

x = the position along the wave profile that corresponds to the angular position $\theta$ and is measured in the direction to the wave propagation starting from the crest (see Figure IV C-2).

From equation IV C-1.15 the surface position at which the maximum force or any force occurs can be calculated. If the force at a particular position along the wave profile is to be calculated, equation IV C-1.11 is used to calculate the angular position of the particle, $\theta$, which is then substituted in equation IV C-1.7, 1.8, 1.9, 1.10, 1.12 or similar type equations from which the desired results are calculated. Figure IV C-5 is a graph of all values of $x/L$, $\theta$ and $H/2L \tanh (2\pi d/L)$ to simplify the labor of calculation.

## 2. Numerical Examples of Wave Force On a Small Object Fixed Below the Surface of a Wave:

### a. The Horizontal Force on a Sphere:

Suppose a sphere of diameter D = $1\frac{1}{2}$ feet has its centerline fixed S = 6 feet from the bottom in water of depth d = 40 feet and is subject to a 6 foot wave (H) with period, T = 6 seconds.

(i) Compute, wave length

$$L_o = 5.12T^2 = 185 \text{ feet (deep water wave length)}$$

$$\frac{d}{L_o} = \frac{40}{185} = 0.216$$

From "Oscillatory Waves", Bulletin of Beach Erosion Board, Special Issue No. 1, July 1, 1948 (Reference IV C-13) or Section II B of this Manual.

$$\frac{d}{L} = 0.239$$

$$L = \frac{d}{0.239} = \frac{40}{0.239} = 167 \text{ feet}$$

$$\frac{H}{L} = \frac{6}{167} = 0.036 \text{ (wave steepness)}$$

(ii) Computed Reynolds number based on centerline velocity from Equation IV C-1.11

$$Re = \frac{Du}{\nu}$$

where

$$D = 1.50 \text{ feet}$$

$$\nu = 10^{-5} \frac{ft^2}{sec} \text{ (kinematic viscosity of salt water)}$$

$$u = \frac{\pi H}{T} \frac{\text{Cosh } \frac{2\pi S}{L}}{\text{Sinh } \frac{2\pi d}{L}} \text{ (maximum horizontal velocity at elevation S)}$$

$$\frac{S}{L} = \frac{6}{167} = 0.036$$

$$u = \frac{\pi 6.0}{6.0} \frac{1.026}{2.133} = 1.51 \text{ ft/sec}$$

and then

$$Re = \frac{1.5 \times 1.51}{10^{-5}} = 2.3 \times 10^5$$

From Figure IV C-4

$$C_d = 0.47$$

and from Table IV C-2

$$C_M = 1.59$$

(iii) Compute angular position of maximum horizontal force from equation (IV C-1.14)

$$\text{Sin } \beta = \frac{2 C_M V_m}{C_d A_p H K}$$

$$V_m = \frac{\pi D^3}{6} = \frac{\pi (1.5)^3}{6} = 1.77 \text{ feet}^3 \text{ (volume of sphere)}$$

$$A_p = \frac{\pi D^2}{4} = \frac{\pi (1.5)^2}{4} = 1.77 \text{ feet}^2 \text{ (projected area of sphere)}$$

$$K = \frac{\text{Cosh } \frac{2\pi S}{L}}{\text{Sinh } \frac{2\pi d}{L}} = \frac{1.026}{2.133} = 0.48 \text{ (equation IV C-1.13)}$$

$$\text{Sin } \beta = \frac{2 \times 1.59 \times 1.77}{0.47 \times 1.77 \times 6 \times 0.48} = 2.35 \text{ (cannot be greater than 1.0)}$$

$$\therefore \beta = 90^\circ$$

(iv) Compute maximum force from equation (IV C-1.12) where the first term is zero when $\beta = 90^\circ$

$$F_H = + 2\pi^2 \rho_w C_M V_m \frac{H}{T^2} K \text{ Sin } \theta$$

$$= + 2\pi^2 (2) 1.59 (1.77) \frac{6}{(6)^2} (0.48) (1.0) \text{ lbs}$$

$$F_H = 8.9 \text{ lbs.}$$

b. Discussion of problem: Figure IV C-6 shows a model study time history of the horizontal wave force on a sphere under different conditions than in the example. (Reference IV C-10) The ratio of force to the maximum force was used so that the close agreement of the time history and form of the curves of the theory and the experiment results could be shown. The amplitude of the maximum measured force was 0.00458lbs. as compared to 0.00402 and 0.00420 pounds as calculated by the theory. The coefficient of drag and mass used to compute the theoretical curve were computed from the measured curve. Two values of each coefficient were computed and their average values used in calculating the theoretical curve. Figure IV C-6 shows that the phase angle is nearly $90^{o}$ as in the example.

The horizontal force on a horizontal cylinder (cross member) at a known elevation above the bottom is computed in a similar manner to that shown above. Figure IV C-7 (Reference IV C-7) shows the model study time history of the horizontal wave force on a horizontal cylinder (where $F_T$ is the total force amplitude). In this case two theoretical curves were plotted, one using the average values of $C_d$ and $C_M$ as in Figure IV C-6. The other curve was plotted using the coefficient of drag computed from the force measured as the trough passed ($\theta = 180^{o}$) and the coefficient of mass at the particle angular position $\theta = 90^{o}$. The maximum (negative) amplitude of the measured force at the trough was 0.056 lbs. and the maximum theoretical (negative) force was 0.056 and 0.058 lbs.

c. The Vertical Force on a Horizontal Cylinder: Suppose a cylinder D = 0.75 feet in diameter is fixed with its centerline, S = 34 feet from the bottom in water of depth d = 40 feet and is subject to a wave of height, H = 6 feet a period of T = 6 second. The long axis of the cylinder is perpendicular to the wave direction. The length of the cylinder is 8.75 feet giving a $D/\ell = 1/5$.

(i) The length is L = 167 feet, computed as before
$H/L = 0.036$
$d/L = 0.239$

(ii) Compute Reynolds number from equation IV C-1.11

$$Re = \frac{Dv}{\nu}$$

$D = 0.75$ feet

$\nu = 10^{-5}$ ft$^2$/sec (kinematic viscosity)

$$v = + \frac{\pi H}{T} \frac{\text{Sinh} \frac{2\pi S}{L}}{\text{Sinh} \frac{2\pi d}{L}}$$

$$\frac{S}{L} = \frac{34}{167} = 0.204$$

$$v = \frac{\pi 6.0}{6} \frac{1.663}{2.133} = 2.45 \text{ ft/sec}$$

and then

$$Re = \frac{0.75\ (2.45)}{10^{-5}} = 1.8 \times 10^{5}$$

From Figure IV C-4

$$C_d = 0.77$$

and from Table IV C-2

$$C_M = 0.98$$

(iii.) Compute angular position of maximum vertical force. From equations IV C-1.6, 1.8 and 1.10 by substitution we have

$$F_v = \pm \frac{\pi^2}{2} \rho_w C_d A_p \frac{H^2}{T^2} K_1^2 \sin^2\theta - 2\pi^2 \rho_w C_M V_m \frac{H}{T^2} K_1 \cos\theta \qquad \text{(IV C-2.1)}$$

and computing angular position of maximum as before we have

$$\cos\beta = \frac{2 C_M V_m}{C_d A_p K_1 H} \qquad \text{(IV C-2.2)}$$

It should be noticed that the plus and minus are used as the -Sin $\theta$ in this case. Now consider only 1 foot or unit length of the cylinder.

$$V_m = \frac{\pi D^2}{4} \times 1 = \frac{\pi (0.75)^2}{4} = 0.44 \text{ feet}^3 \text{ (volume of cylinder one foot long)}$$

$$A_p = D \times 1 = 0.75 \text{ feet}^2 \text{ (projected area of one foot of cylinder)}$$

$$K_1 = \frac{\sinh \frac{2\pi S}{L}}{\sinh \frac{2\pi d}{L}} = \frac{1.663}{2.133} = 0.78 \qquad \text{(IV C-2.3)}$$

$$\cos\beta = - \frac{2 \times 0.98 \times 0.44}{0.77 \times 0.75 \times 0.78 \times 6} = -0.332$$

$$\beta = 109^\circ 23' \quad (\sin\beta = 0.943)$$

(iv) Compute maximum vertical force from equation (IV C-2.1) where $\pm (-0.943)^2$ is plus

$$F_v = + \frac{\pi^2}{2} \quad 2.00 \times 0.77 \times 0.75 \frac{6^2}{6^2} \quad (0.78)^2 (+0.943)^2$$

$$-2\pi^2 \, (2.00) \times 0.98 \times (0.44) \frac{6}{6^2} \quad (0.78) \, (-0.332)$$

$$= +5.20 + 0.73 = 5.93 \text{ lbs/ft}$$

d. Discussion of Problem: Figure IV C-8 (Reference IV C-7) shows a model study time history of the vertical force on a horizontal cylinder under somewhat different conditions than in the above example. The coefficients of drag and mass used to compute the theoretical curves were obtained from the measured force. For one curve the average value of the coefficient of drag and the average values of the coefficient of mass are used. In the other case, the coefficient of drag used was computed from the force when the angular particle position was $\theta = 270^\circ$, and the coefficient

of mass was computed when the trough ($\theta = 180^{\circ}$) passed the cylinder. The maximum measured negative amplitude of force was 0.024 lbs. and the maximum (negative) theoretical force was 0.022 and 0.023 lbs.

3. References:

IV C-1: Birkhoff, Garret. "Hydrodynamics". Princeton University; 1950.

IV C-2: Brahmig, Rolf. "Experimental Determination of the Hydrodynamic Increase in Mass in Oscillating Bodies". Translation 118, The David Taylor Model Basin; Washington, D. C.; U. S. Navy.

IV C-3: Dryden, H. L., F. D. Murnaghan and H. Bateman. "Hydrodynamics". National Research Council; 1932.

IV C-4: Iversen, H. W. and R. Balent. "A Correlating Modulus for Fluid Resistance in Accelerated Motion". Series 3 Issue 314, Institute of Engineering Research, University of California; Berkeley, California.

IV C-5: Lamb, Horace. "Hydrodynamics". Dover, New York, 6th ed; 1945, p 4, 19, and 363.

IV C-6: Marks, Lionel S. "Mechanical Engineers' Handbook". 4th ed. McGraw, Hill, New York; 1941.

IV C-7: Morison, J. R. "The Force Exerted by Waves on Marine Structures". Series 35, Issue 3, Institute of Engineering Research, University of California; Berkeley, California; October 20, 1950.

IV C-8: Morison, J. R., M. P. O'Brien, J. W. Johnson, and S. A. Schaaf. "The Force Exerted by Surface Waves on Piles". Petroleum Transactions. AIME, Vol. 189; 1950.

IV C-9: O'Brien, M. P. and G. H. Hickox. "Applied Fluid Mechanics". McGraw-Hill New York. 1937.

IV C-10: O'Brien, M. P. and J. R. Morison. "The Forces Exerted by Waves on Objects". Series 3, Issue 310, Institute of Engineering Research, University of California; Berkeley, California; August 31, 1950.

VI C-11: Rouse, H. "Fluid Mechanics for Hydraulic Engineers". McGraw, Hill; New York; 1938.

VI C-12: Stokes, Sir G. G. "On the Effect of the Internal Friction of Fluids on the Motion of Pendulums". Mathematical and Physical Papers, Vol. III, Cambridge University Press; 1901.

VI C-13: Stokes, Sir G. G. "On the Theory of Oscillatory Waves". Mathematical and Physical Papers, Vol. V; Cambridge University Press; 1901.

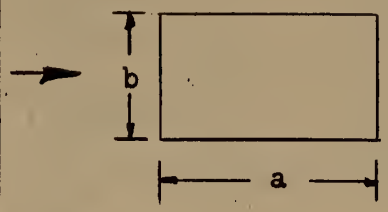
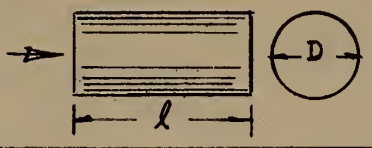
VI C-14: Wiegel, R. L. "Oscillatory Waves". (Tables) The Bulletin of the Beach Erosion Board, Special Issue No. 1; July 1, 1948.

VI C-15: Yee-Tak-Yu. "Virtual Masses and Moments of Inertia of Disks and Cylinders in Various Liquids". Journal of Applied Physics, Vol. 13, No. 1; January 1942.

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TABLE IV C-1

## COEFFICIENT OF DRAG *

| Object | Proportions | Position | $C_d$ | Remarks |
|---|---|---|---|---|
| Rectangular plate, sides a and b (steady flow) | $\frac{a}{b}$ 1<br>4<br>5<br>8<br>12.5<br>20<br>25<br>50<br>$\infty$ |  | 1.16<br>1.17<br>1.20<br>1.33<br>1.34<br>1.50<br>1.57<br>1.76<br>2.00 | $Re > 10^3$ |
| Two disks, spaced a distance apart (steady flow) | $\frac{l}{D}$ 0<br>1<br>1.5<br>2<br>3 |  | 1.12<br>0.93<br>0.78<br>1.04<br>1.52 | $Re > 10^3$ |
| Cylinder (steady flow) | $\frac{l}{D}$ 0<br>1<br>2<br>4<br>7 |  | 1.12<br>0.91<br>0.85<br>0.87<br>0.99 | $Re > 10^3$ |
| Circular disk (steady flow) | |  | 1.11 | $Re > 10^3$ |
| Circular cylinder axis perpendicular to flow (steady flow) | 1<br>5<br>20<br>$\infty$<br>5<br>$\infty$ | | 0.63<br>0.74<br>0.90<br>1.20<br>0.35<br>0.33 | $Re > 10^5$<br><br><br><br>$Re > 5 \times 10^5$ |
| Streamlined foil 1:3 airplane strut (steady flow) | $\infty$ | | 0.10 | $Re > 5 \times 10^4$ |
| Sphere (steady flow) | | | 0.47<br>0.20 | $Re > 10^5$<br>$Re > 3 \times 10^5$ |
| Ellipsoid - 1/3 maj. axis parallel to flow (steady flow) | | | 0.06 | $Re > 2 \times 10^5$ |

* References IV C-3, 4, 6, 7, 8, 9, 10, 11 and 15.

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TABLE IV C-1 (cont.)

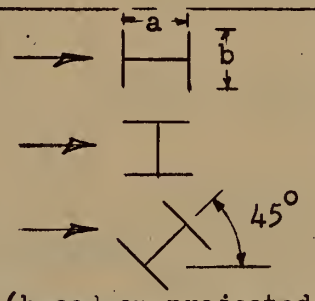
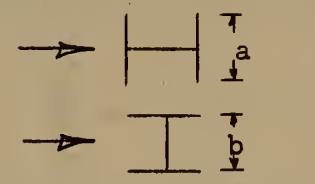
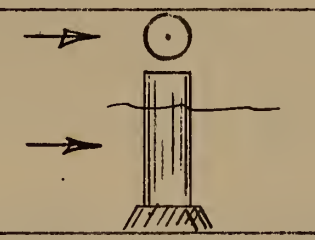
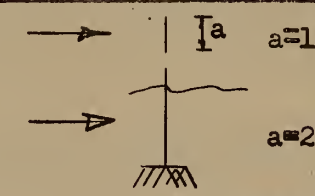
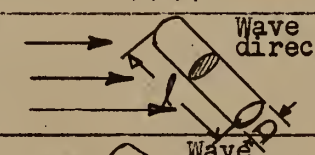
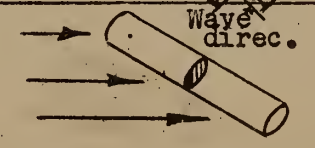
| Object | Proportions | Position | $C_d$ | Remarks |
|---|---|---|---|---|
| Airship hull model (steady flow) | | | 0.042 | Re $2 \times 10^5$ |
| Hemispherical cup, open back (steady flow) | |  | 0.41 (0.34) | Re $11^3$ |
| Hemispherical cup, open front (steady flow) | |  | 1.35 (1.33) | Re $11^3$ |
| Cone, closed base (steady flow) | |  | $\alpha = 60°$ 0.51<br>$= 30°$ 0.34 | |
| H-section pile (steady flow) | $\frac{a}{b} = 1$ |  (based on projected area ads) | 1.5<br>1.6<br><br>1.5<br>1.6<br><br>1.9<br>2.0 | $4.3 \times 10^4$<br>$3.0 \times 10^4$<br><br>$4.3 \times 10^4$<br>$3.0 \times 10^4$ Re<br><br>$4.3 \times 10^4$<br>$3.0 \times 10^4$ |
| H-section pile (oscillating flow, overall coeff.) | $\frac{a}{b} = 1$ |  | 2.01±0.38<br><br>1.78±0.22 | 1.0 to $1.5 \times 10^4$ Re<br><br>1.0 to $1.5 \times 10^4$ |
| Round pile (oscillating flow, overall coeff.) | |  | 1.63±0.41 | $2.2 \times 10^{+3}$ to $1.1 \times 10^4$ Re |
| Flat plate pile (oscillating flow, overall coeff.) | |  a=1" a=2" | 1.29±0.44<br><br>3.16±1.75 | 1.0 to $1.5 \times 10^4$ Re<br><br>2.0 to $3.0 \times 10^4$ |
| Round cylinder (oscillating flow) horizontal force | $\frac{D}{\ell} = 1/2.5$, $1/5$, $1/10$ |  Wave direc. | 1.43±0.49 | 1.0 to $1.4 \times 10^4$ Re |
| Round cylinder (oscillating flow) vertical force | $\frac{D}{\ell} = 1/5$ |  Wave direc. | 2.63±1.20 | 1.0 to $1.2 \times 10^4$ Re |

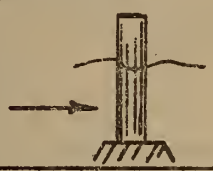
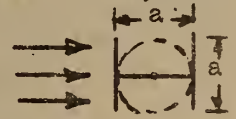
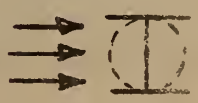
TABLE IV C-2

COEFFICIENT OF MASS *

| Investigator | Shape of Object | Type of Motion | Fluid | $C_M$ | Remarks |
|---|---|---|---|---|---|
| Bessel | Sphere | Oscillating | Air<br>Water | 1.9<br>1.6 | Experiments with spherical pendulum |
| Poisson | Sphere | Unidirectional | Ideal | 1.5 | Analytical evaluation from potential flow |
| Stokes | Sphere | Unidirectional | Viscous | 1.5+ | Analytical evaluation with laminar flow |
| McEwen | Sphere | Oscillating | Viscous | 1.5+ | Experimental |
| Krishnayer | Sphere | Oscillating | Water | 1.58 | Experimental |
| Lunnon | Spheres | Unidirectional | Water | 1.5 to 3.0 | Experimental |
| Morison | Spheres | Oscillating | Water | 1.59 $\pm$ 0.30 | Re $1.0 \rightarrow 1.4 \times 10^4$ Experimental |
| Smith | Elongated body of revolution | Unidirectional | Air | 1.072 | Study of airship - Akron |
| Taylor | Elongated body of revolution | Linear | Ideal | 1.051 | Airship hull shape from potential flow |
| Browne, Moullin & Perkins | Rectangular bars | Vertical oscillation | Water | 1.65 | Experimental with floating prisms |
| Murphy | Ellipsoids and ship models | Unidirectional | Water | 1.03 to 1.07 | Referred to displaced mass of water |
| Yee-Tak-Yu | Disk | Oscillating | Water, gasoline & carbon tetrachloride | 1.81 | Experimental |

* References IV C-3, 4, 7, 8, 10, 12 and 15.

TABLE IV C-2 (cont.)

| Investigator | Shape of Object | Type of Motion | Fluid | $C_M$ | Remarks |
|---|---|---|---|---|---|
| Lamb | Disk | Perpendicular to plane of disk | Ideal | 1.637 | Referred to volume of a sphere with diam. equal to disk diameter |
| Gracey | Flat plates | Oscillating | Air | 1.94 and 1.96 | Pendulum oscillating with different length, breadth ratio |
| Morison | Flat plate pile  | Oscillating | Water | 0.72 ± 0.29 (Re 1.0-1.5 x $10^4$)<br>1.20 ± 0.63 (Re 2.0-3.0 x $10^4$) | Experimental: {1" width, 2" width} Calibration based on volume of equal diameter cylinder |
| Morison | Cylinder | Oscillating | Water | 0.98 ± 0.51 (Re 1.0-1.4 x $10^4$) | Experimental (model wave)  |
| Morison | Cylindrical pile | Oscillating | Water | 1.51 ± 0.20 $2.2 \times 10^3$-$2.3 \times 10^4$ | Experimental  |
| Morison | H-section pile | Oscillating | Water | 2.59 ± 1.19 | Experimental Re 1.0-1.5 $10^4$  |
| | | | | 3.05 ± 0.96 | Calculation based on volume of equal diameter cylinder  |

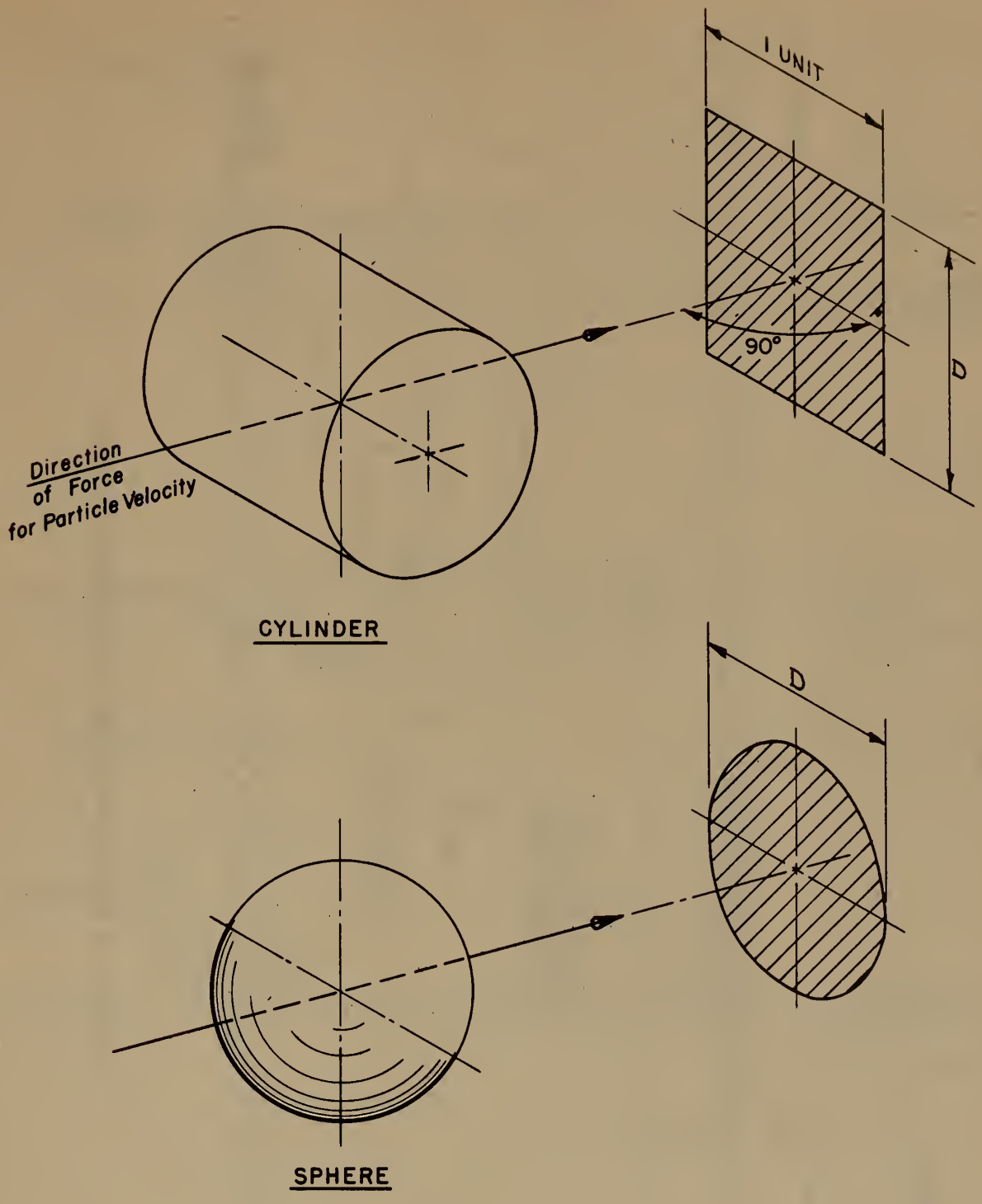



FIG. IVC-1 -- PROJECTED AREA

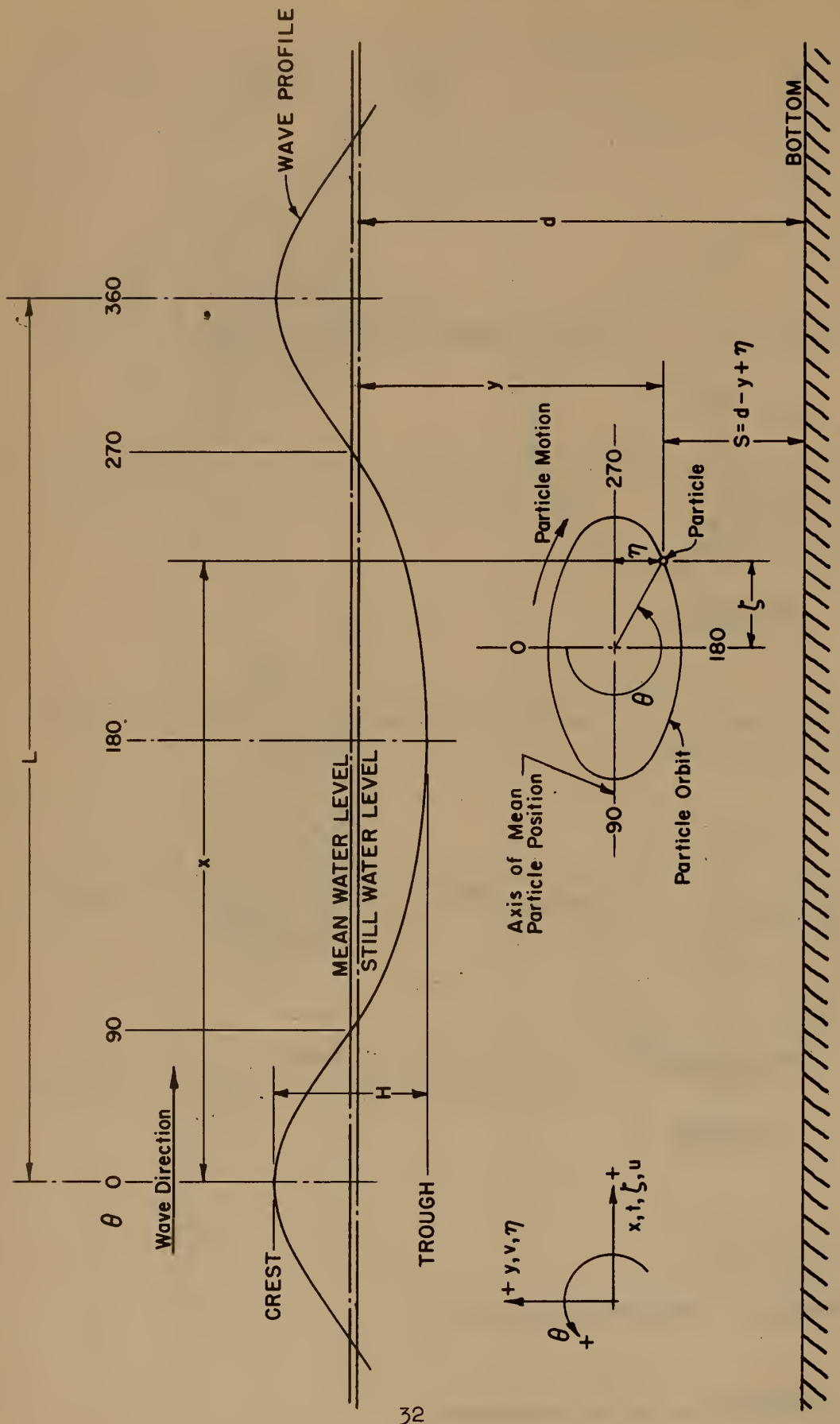



FIG. IV C-2 -- WAVE AND ORBIT NOMENCLATURE

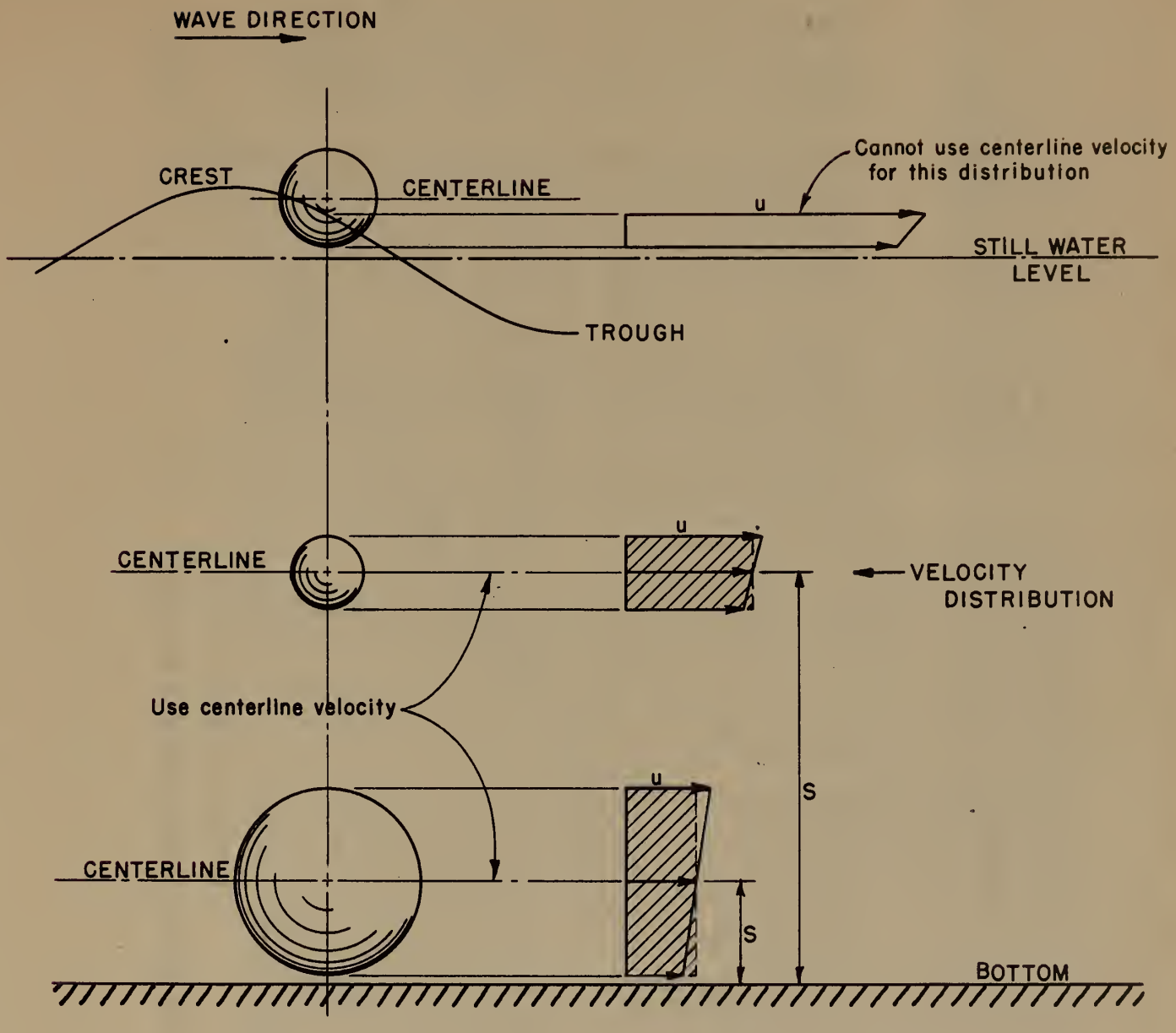



FIG. IV C - 3 -- WATER PARTICLE VELOCITY DISTRIBUTIONS AND CENTERLINE VELOCITY.

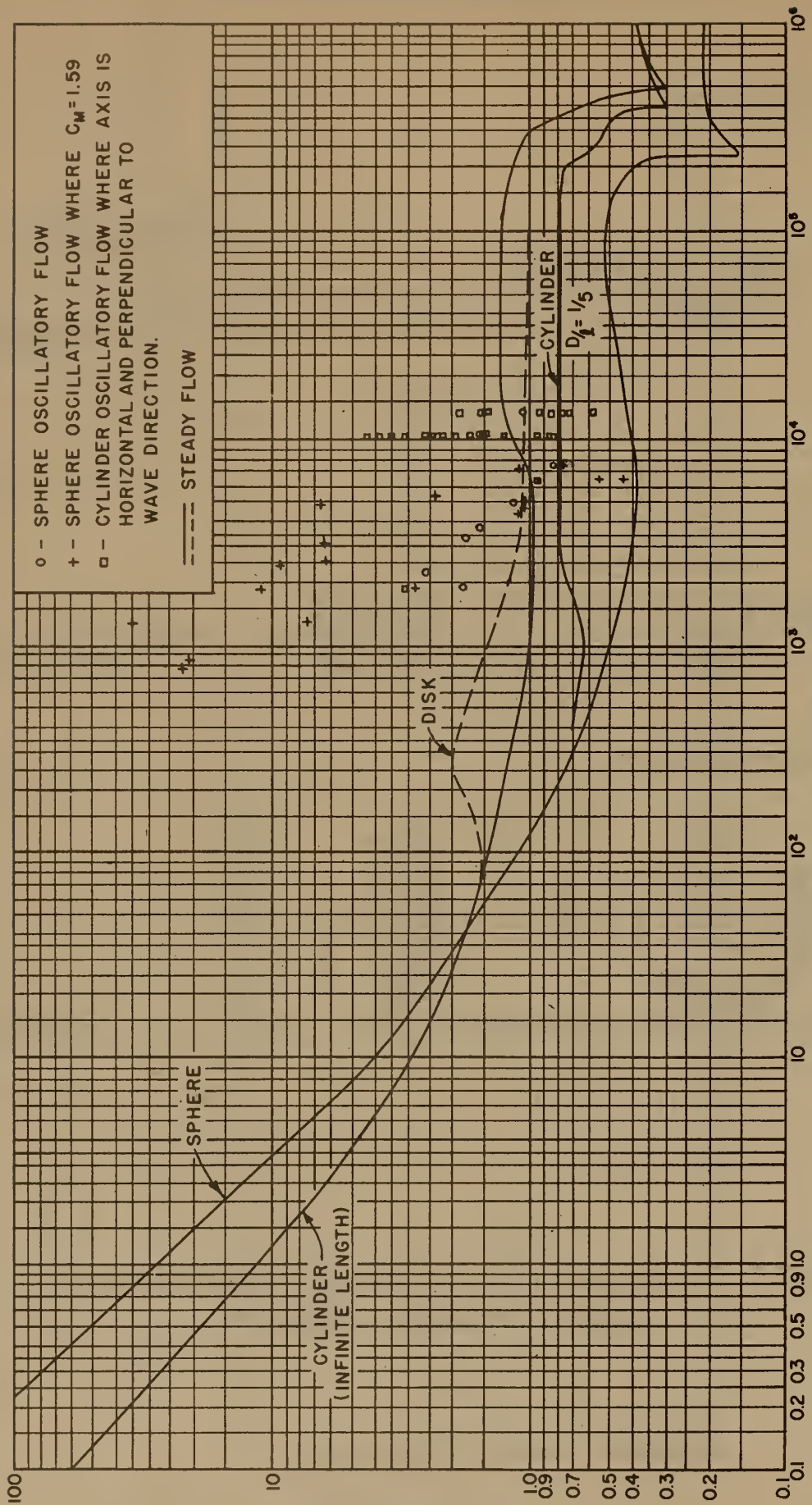



FIG. IV C-4 -- COEFFICIENTS OF DRAG

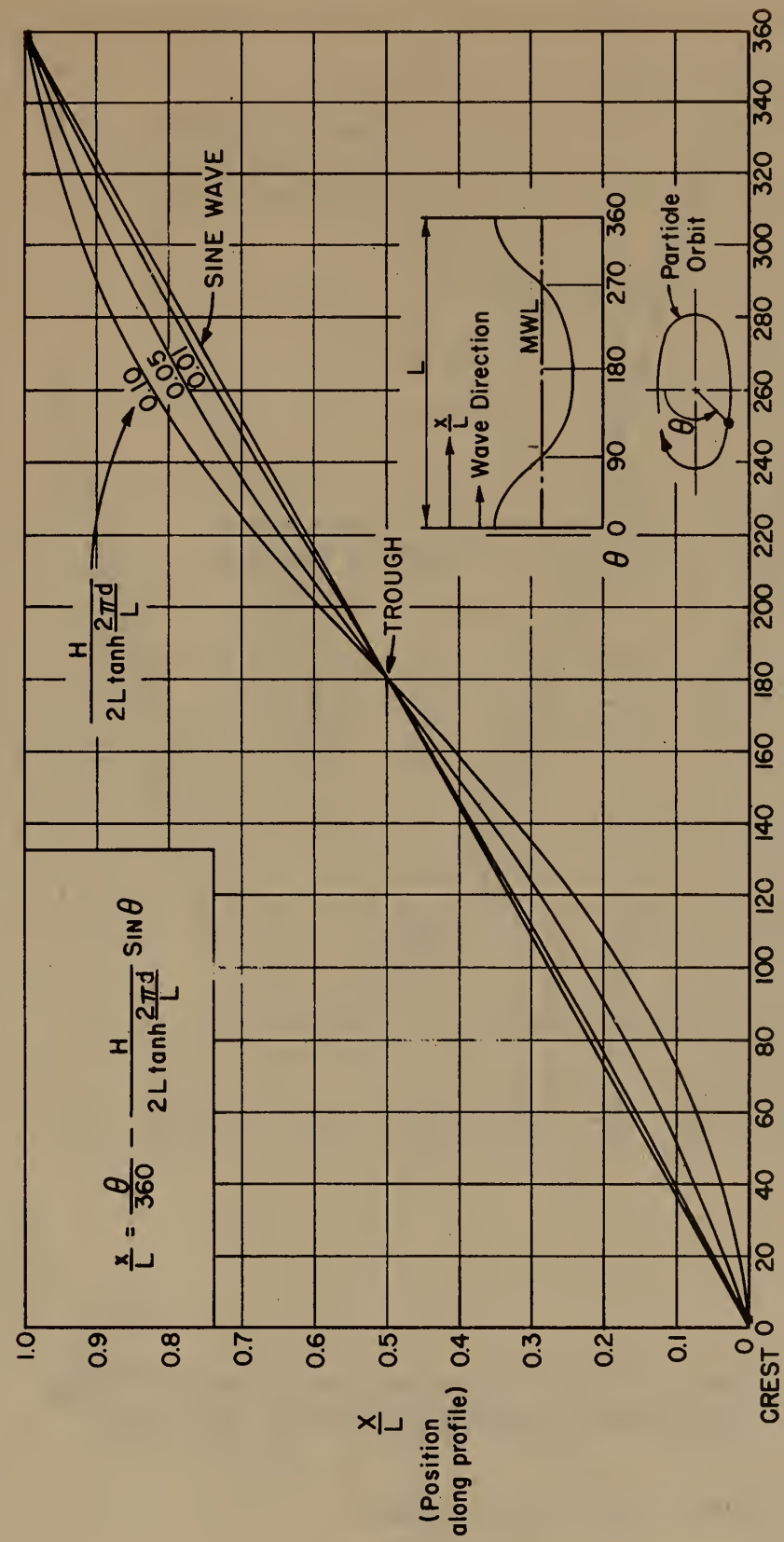



FIG. IV C-5 -- RELATION OF SURFACE POSITION TO ANGULAR POSITION

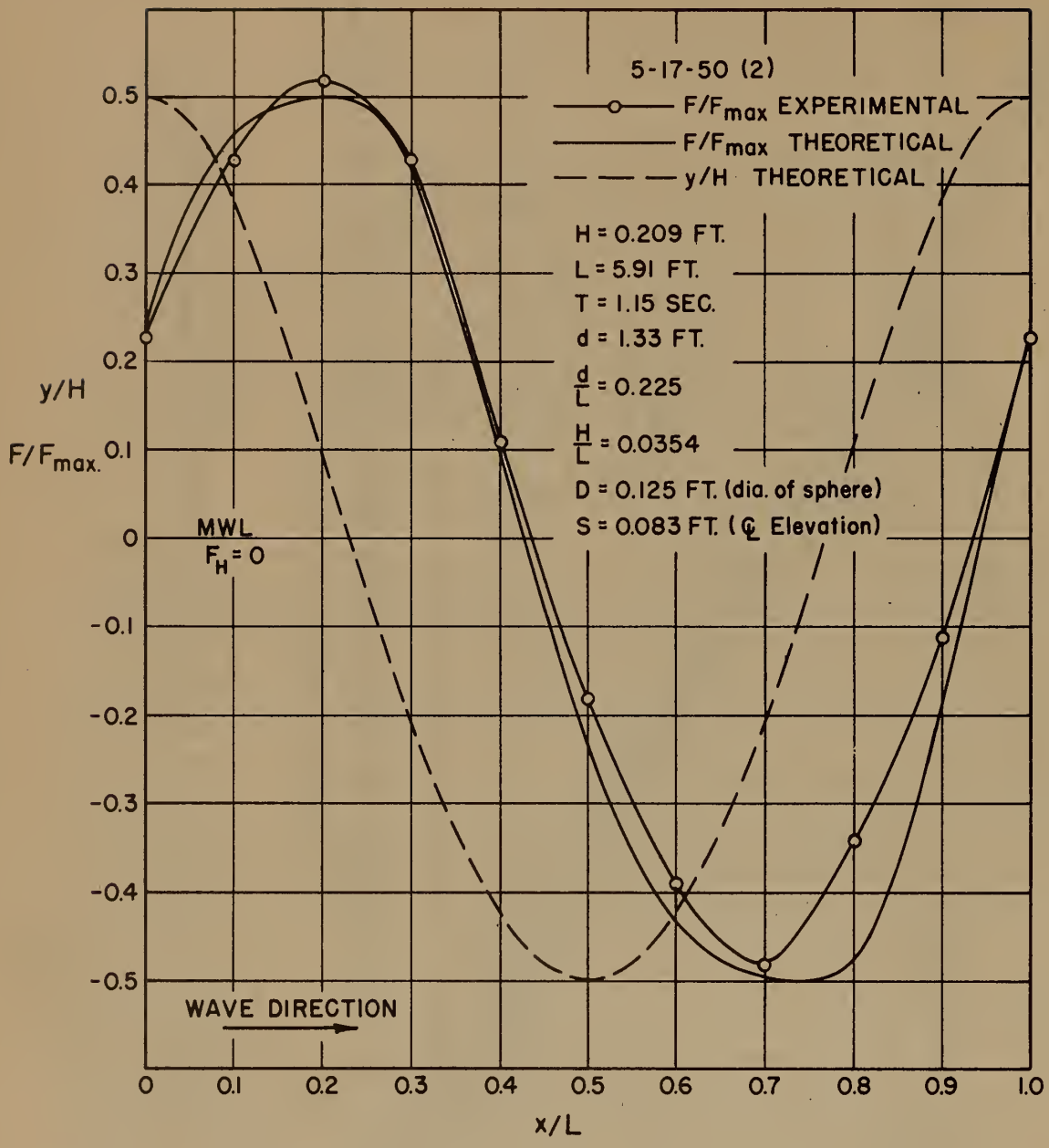



FIG. IVC-6 -- TIME HISTORY OF HORIZONTAL WAVE FORCE ON SPHERE SUSPENDED NEAR BOTTOM.

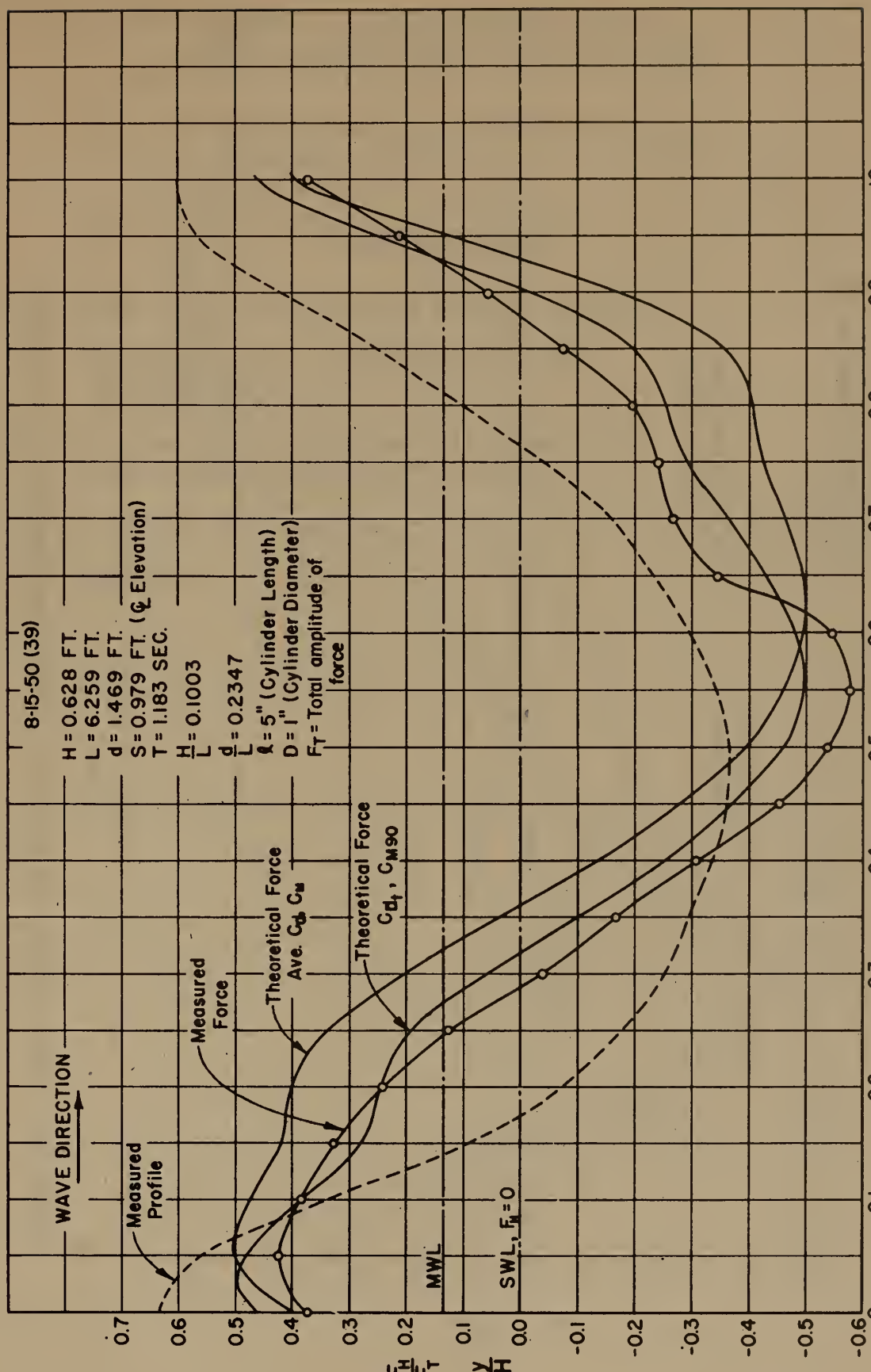



FIG. IIC-7 - TIME HISTORY - HORIZONTAL FORCE OF STEEP WAVE ON TRANSVERSE HORIZONTAL CYLINDER.

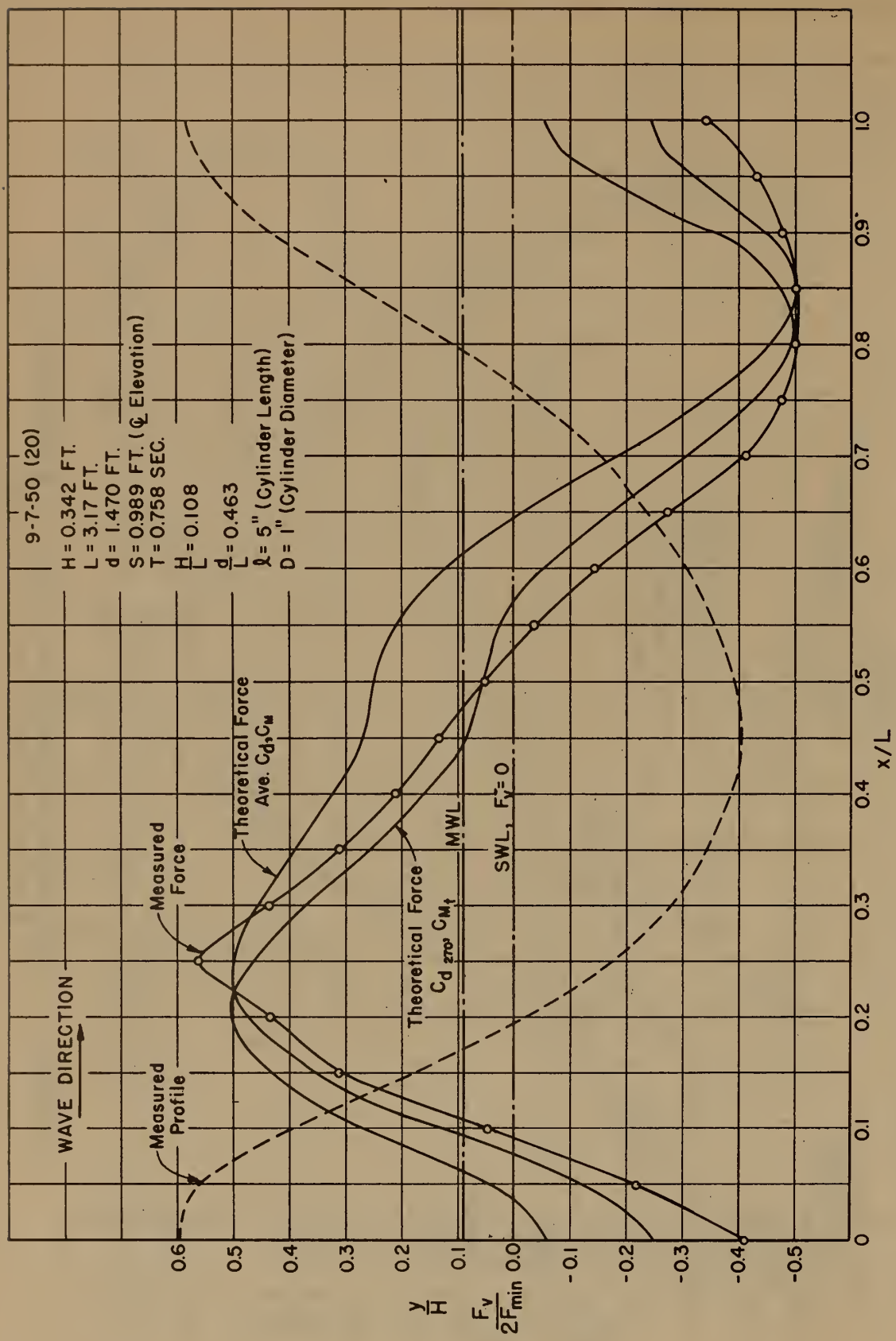



FIG. IV C-8 -- TIME HISTORY OF VERTICAL FORCE ON HORIZONTAL CYLINDER.

# MANUAL OF AMPHIBIOUS OCEANOGRAPHY

## SECTION IV: WAVE FORCES ON OBJECTS, PILING, VERTICAL WALLS AND RUBBLE MOUNDS

### D: FORCES ON LARGE OBJECTS AND PILING

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| 2. | The Total Force on a Submerged Section of a Single Round Pile | 3 |
| 3. | The Total Force on a Section of Round Piling that Extends above the Wave Surface | 4 |
| 4. | Discussion of Total Force | 6 |
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## D. FORCES ON LARGE OBJECTS AND PILING

1. Forces on Large Objects and Sections of Piles -- This section deals with the extension of the theory given in the previous section to larger objects and in particular to sections of piling and to objects and piling that extend through the surface of the wave. The extension is made by integrating the differential force over the vertical height of the object since the particle velocity and acceleration are functions of the vertical position (S). (Reference IV D-1) When the upper limit of the integral is the water surface then S is given by the expression

$$S_s = d + \frac{H}{2}\left\{\cos\frac{2\pi x}{L} + \frac{\pi H}{2L}\left[\frac{\cosh\frac{2\pi d}{L}\left(\cosh\frac{4\pi d}{L}+2\right)}{2\left(\sinh\frac{2\pi d}{L}\right)^3}\right]\cos\frac{4\pi x}{L}\right\} \quad \text{(IV D-1.1)}$$

where x is given by equation IV C-1.15. For simplification of calculations the coefficient of the second term of equation IV D-1.1 is plotted in Figure IV D-1.

The changes that occur in equation IV C-1.6 for the force are that the equation now represents an increment of force, that $A_p$ is replaced by an increment of projected area and, that $V_m$ is replaced by an increment of volume. It is obvious that when the object or pile extends above the surface of the wave that no simple solution exists for the maximum force and the position of the maximum force. These quantities are determined readily by a time history graph of the particular problem. It is, of course, possible to make a few simplifying assumptions, and thus obtain the approximate position of the maximum and thence the exact force at that position; however, this process is as long and tedious a task as is the time history graph. As before the equations are valid in depths of water where d/L is greater than 0.20.

In considering piles of shapes other than round, for example, H sections and flat plates, the increment of projected area has the same definition as before but the increment of volume must be given a new definition. The necessity of this change is readily seen when we consider the volume of water displaced by an H-Section of 1/8" wall thickness as compared to the volume where the wall is 1/4" thick. The volume has been doubled but the flow pattern has not been altered materially and hence no noticable change will occur in the virtual mass term. Thus, the increment of volume is taken as a cylindrical volume with diameter equal to the width of the projected area or the characteristic length. This definition is shown in Figure IV D-2.

For the particular problem of the horizontal force on a section of a piling, the expression of the increment of force from equation IV C-1.6 becomes

$$dF_H = \tfrac{1}{2}\rho_w C_d u^2 D\, dS + \rho_w C_M \frac{\partial u}{\partial t}\frac{\pi D^2}{4} dS \quad \text{(IV D-1.2)}$$

and the total thrust over a section $S_1$ to $S_2$ is obtained by integrating equation IV D-1.2.

$$F_H = \int_{S_1}^{S_2} dF_H \quad \text{(IV D-1.3)}$$

where $S_1$ is less than $S_2$ but both $S_1$ and $S_2$ can have any value of S from 0 to $S_s$. The definition of these symbols are shown in Figure IV C-3. Now substitute equation IV C-1.7 and 1.9 into equation IV D-1.2 and then substitute equation

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IV D-1.2 into equation IV D-1.3 to obtain the expression

$$F_H = \int_{S_1}^{S_2} \frac{\pi^2}{2} \rho_w C_d D \frac{H^2}{T^2} \frac{\left(\text{Cosh} \frac{2\pi S}{L}\right)^2}{\left(\text{Sinh} \frac{2\pi d}{L}\right)^2} \text{Cos}^2\theta \, dS$$

$$+ \int_{S_1}^{S_2} \frac{\pi^3}{2} \rho_w C_M D^2 \frac{H}{T^2} \frac{\text{Cosh} \frac{2\pi S}{L}}{\text{Sinh} \frac{2\pi d}{L}} \text{Sin}\,\theta \, dS \qquad \text{(IV D-1.4)}$$

and

$$F_H = +\pi \rho_w C_d D \frac{H^2 L}{T^2} k_1 \text{Cos}^2\theta + \frac{\pi^2}{4} \rho_w C_M D^2 \frac{HL}{T^2} k_2 \text{Sin}\,\theta \qquad \text{(IV D-1.5)}$$

where from the integration the values of $k_1$ and $k_2$ are

$$k_1 = \frac{\frac{4\pi S_2}{L} - \frac{4\pi S_1}{L} + \text{Sinh} \frac{4\pi S_2}{L} - \text{Sinh} \frac{4\pi S_1}{L}}{16 \left(\text{Sinh} \frac{2\pi d}{L}\right)^2} \qquad \text{(IV D-1.6)}$$

$$k_2 = \frac{\text{Sinh} \frac{2\pi S_2}{L} - \text{Sinh} \frac{2\pi S_1}{L}}{\text{Sinh} \frac{2\pi d}{L}} \qquad \text{(IV D-1.7)}$$

If $S_2$ is less than $S_s$ then the position of the maximum thrust is obtained by differentiating equation IV D-1.5

$$\text{Sin}\beta = \frac{\pi C_M D k_2}{8 C_d H k_1} \qquad \text{(IV D-1.8)}$$

where $\beta$ is defined as the angular position of the particle for the maximum thrust. The value $\beta$ is a value of $\theta$.

The maximum thrust is computed by substituting the value of $\sin\beta$ and $\cos\beta$ into equation IV D-1.5 which gives the expression

$$F_H = \rho_w D \frac{H^2 L}{T^2} \left\{ \pi C_d k_1 \left[ 1 + \left( \frac{\pi C_M D k_2}{8 C_d H k_1} \right)^2 \right] + \frac{\pi^2}{4} \frac{D}{H} C_M k_2 \left( \frac{\pi C_M D k_2}{8 C_d H k_1} \right) \right\} \qquad \text{(IV D-1.9)}$$

However, if $S_2 = S_s$ in equation IV D-1.5 and since $S_s$ is a transcendental function of $\theta$, an explicit expression for $\beta$ is impossible and the position of maximum force must be found by successive approximation or by graph. Since specific problems are generally encountered, the easiest method is to plot the time history of $F_H$ and from the position of the maximum force along the profile, the angle $\beta$ can be computed. Examples of each of the



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cases mentioned are given below.

2. The Total Force on a Submerged Section of a Single Round Pile -- Suppose a round pile of diameter, D = 0.75 ft., is fixed at the elevation $S_1$ = 2 ft. and $S_2$ = 32 ft. from the bottom in a total depth of water d = 40 ft. The pile is subject to the force exerted by a wave of height H = 6 ft. and having a period of T = 6 sec.

(i) Wave Length L = 167 ft. as previously calculated (IV C-2a)

$$\frac{d}{L} = 0.239 \qquad \frac{S_1}{L} = 0.012$$

$$\frac{H}{L} = 0.036 \qquad \frac{S_2}{L} = 0.192$$

(ii) Compute Reynold's number based on the maximum horizontal velocity at $S_2$

$$u = \frac{\pi H}{T} \frac{\text{Cosh } \frac{2\pi S_2}{L}}{\text{Sinh } \frac{2\pi d}{L}} = \frac{\pi 6}{6} \times \frac{1.820}{2.133} = 2.69$$

$$Re = \frac{D u}{\nu} = \frac{0.75 \times 2.69}{10^{-5}} = 2.0 \times 10^5$$

From Figure IV C-4 $C_d = 1.20$ (Steady flow value for infinite cylinder)

and from Table IV C-2 $C_M = 1.51$

(iii) Compute angular position of total horizontal force from equation IV D-1.8. First caluclate $k_1$ (equation IV D-1.6) and $k_2$ (equation IV D-1.7)

$$k_1 = \frac{\frac{4\pi S_2}{L} - \frac{4\pi S_1}{L} + \text{Sinh } \frac{4\pi S_2}{L} - \text{Sinh } \frac{4\pi S_1}{L}}{16 \left(\text{Sinh } \frac{2\pi d}{L}\right)^2}$$

$$= \frac{2.413 - 0.1508 + 5.538 - 0.1513}{16 (2.133)^2} = 0.105$$

$$k_2 = \frac{\text{Sinh } \frac{2\pi S_2}{L} - \text{Sinh } \frac{2\pi S_1}{L}}{\text{Sinh } \frac{2\pi d}{L}} = \frac{1.521 - 0.07547}{2.133} = 0.678$$

$$\text{Sin}\beta = \frac{\pi C_M D k_2}{8 C_d H k_1} = \frac{\pi 1.51 \times 0.75 \times 0.678}{8 \times 1.20 \times 6 \times 0.105}$$

$$= 0.339$$

$$\beta = 19^\circ 49' \qquad \text{Cos}\beta = 0.941$$

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(iv) Compute the maximum horizontal force from equation IV D-1.5

$$F_H = \pi \rho_w \frac{DH^2L}{T^2} \left\{ C_d k_1 \cos^2\beta + \frac{\pi}{4} \frac{D}{H} C_M k_2 \sin\beta \right\}$$

$$= \frac{\pi \times 2.0 \times 0.75 \times 6^2 \times 167}{6^2} \left\{ 1.20 \times 0.105 \times (0.941)^2 \right.$$

$$\left. + \frac{\pi \times 0.75 \times 1.51}{4 \times 6} \times 0.678 \times 0.339 \right\}$$

$$= (787.0) \left\{ 0.112 + 0.060 \right\} = 135.0 \text{ lbs.}$$

3. The Total Force on a Section of Round Piling that Extends Above the Wave Surface:

Suppose a round pile of diameter, D = 0.75 ft. is fixed at the elevation $S_1$ = 2 ft. and extends above the wave surface. The total depth of water, d equals 40 ft. The pile is subject to a wave of height H = 6 ft. and of period, T = 6 sec.

1. Wave Length L = 167 ft. as previously calculated (IV C-2a)

$$\frac{d}{L} = 0.239 \qquad \frac{S_1}{L} = 0.012$$

$$\frac{H}{L} = 0.036$$

2. Compute values of $S_s$ from equation IV D-1.1 and thence $S_s/L$

$$S_s = d + \frac{H}{2} \left\{ \cos \frac{2\pi x}{L} + \frac{\pi H}{2L} \left[ \frac{\cosh \frac{2\pi d}{L} \left( \cosh \frac{4\pi d}{L} + 2 \right)}{2 \left( \sinh \frac{2\pi d}{L} \right)^3} \right] \cos \frac{4\pi x}{L} \right\}$$

From Figure IV D-1 for $\pi H/2L = 0.057$ and $d/L = 0.239$

$$S_s = d + \frac{H}{2} \left\{ \cos \frac{2\pi x}{L} + 0.085 \cos \frac{4\pi x}{L} \right\}$$

$$= 40 + \frac{6}{2} \left\{ \cos \frac{2\pi x}{L} + 0.085 \cos \frac{4\pi x}{L} \right\}$$

For $\frac{2\pi x}{L} = 0^\circ$; $S_s = 40 + 3(1.085) = 43.255$; etc.

We compute $S_s$ and $S_s/L$ for every $15^\circ$ of $2\pi x/L$. (See Table IV D-1)

Now compute Reynold's number based on the maximum horizontal velocity at the crest of the wave $S_s$ = 43.26 and $S_s/L$ = 0.259.

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$$u = \frac{\pi H}{T} \frac{\text{Cosh } \frac{2\pi S_s}{L}}{\text{Sinh } \frac{2\pi d}{L}} = \frac{\pi 6}{6} \frac{2.643}{2.133} = 3.89 \text{ ft/sec}$$

$$Re = \frac{Du}{\nu} = \frac{0.75 \times 3.89}{10^{-5}} = 2.9 \times 10^5$$

From Figure IV C-4 $C_d = 1.00$
and from Table IV C-2 $C_M = 1.51$

3. Compute the time history of the horizontal thrust. First we compute $k_1$ and $k_2$ for each $S_s/L$ from the expression

$$k_1 = \frac{\frac{4\pi S_s}{L} - \frac{4\pi S_1}{L} + \text{Sinh } \frac{4\pi S_s}{L} - \text{Sinh } \frac{4\pi S_1}{L}}{16 \left(\text{Sinh } \frac{2\pi d}{L}\right)^2}$$

For $\frac{2\pi x}{L} = 0^o$

$$k_1 = \frac{3.255 - 0.1508 + 12.94 - 0.1513}{16 (2.133)^2} = 0.218$$

$$k_2 = \frac{\text{Sinh } \frac{2\pi S_s}{L} - \text{Sinh } \frac{2\pi S_1}{L}}{\text{Sinh } \frac{2\pi d}{L}}$$

For $\frac{2\pi x}{L} = 0^o$

$$k_2 = \frac{2.447 - 0.07547}{2.133} = 1.111$$

etc. See Table IV D-1

For each $15^o$ of $2\pi x/L$ we obtain values of $x/L$ and from Figure IV C-5 we get the values of $\theta$ where

$$\frac{x}{L} = \frac{\theta}{360} - \frac{H}{2L \text{ Tanh } \frac{2\pi d}{L}} \text{ Sin } \theta$$

and

$$\frac{H}{2L \text{ Tanh } \frac{2\pi d}{L}} = \frac{6}{2 \times 167 \times 0.9055} = 0.020$$

Then each pair of values of $k_1$ and $k_2$ and each $\theta$ is substituted into the force equation

$$F_H = \pi \rho_w \frac{DH^2 L}{T^2} \left\{ \pm C_d k_1 \text{ Cos}^2 \theta + \frac{\pi}{4} \frac{D}{H} C_M K_2 \text{ Sin } \theta \right\}$$

$$= \frac{\pi \times 2.0 \times 0.75 \times 6^2 \times 167}{6^2} \left\{ 1.00 \times k_1 \cos^2\theta + \frac{\pi \times 0.75 \times 1.51}{4 \times 6} \; k_2 \sin\theta \right\}$$

For $\theta = 0^\circ$

$F_H = (787.0) \left\{ 1.00 \times 0.218 \times (1)^2 + 0 \right\}$

$= 171$ lbs.

etc. See Table IV D-1

Now the values of $F_H$ are plotted versus $\theta$ to determine the position and amplitude of the maximum force, see Figure IV D-4.

Maximum $F_H = 192$ lbs.

and $\beta = 24^\circ$

Figure IV D-5 is a plot of $F_H$ and $S_s$ versus $\frac{x}{L}$ which is the theoretical time history and wave profile.

4. Discussion of Total Force -- The effects of varying the pile size, the wave characteristics and the water depth on the maximum force and the angular position are found by examining equations IV D-1.5 and 1.8. If the pile diameter is increased, the force increases and in particular the inertia component increases more than the drag component. The position of maximum force is thus further in advance of the approaching crest. If the wave height is increased the force increases and, in particular, the drag component increase more rapidly than the inertia component and the position of maximum force becomes closer to the position of the crest of the wave. The increase of the wave period increases the wave length, thus decreases the value of d/L. This change effects the values of $k_1$ and $k_2$ which increase or remain constant for decreasing d/L (See Figure IV D-6). Thus, since L is proportional to $T^2$, the force increases somewhat with an increase of wave length. Since $k_2$ is a constant and $k_1$ continues to increase the drag component becomes more important and the position of maximum force occurs closer to the wave crest. The coefficients of drag and mass shown in Figure IV C-4 and Tables IV C-1 and 2 are taken as constants over a large range of wave conditions so long as there is no appreciable change in the Reynold's number. A decrease of water depth decreases the value of d/L and the effect described for the increased wave length is repeated.

5. References:

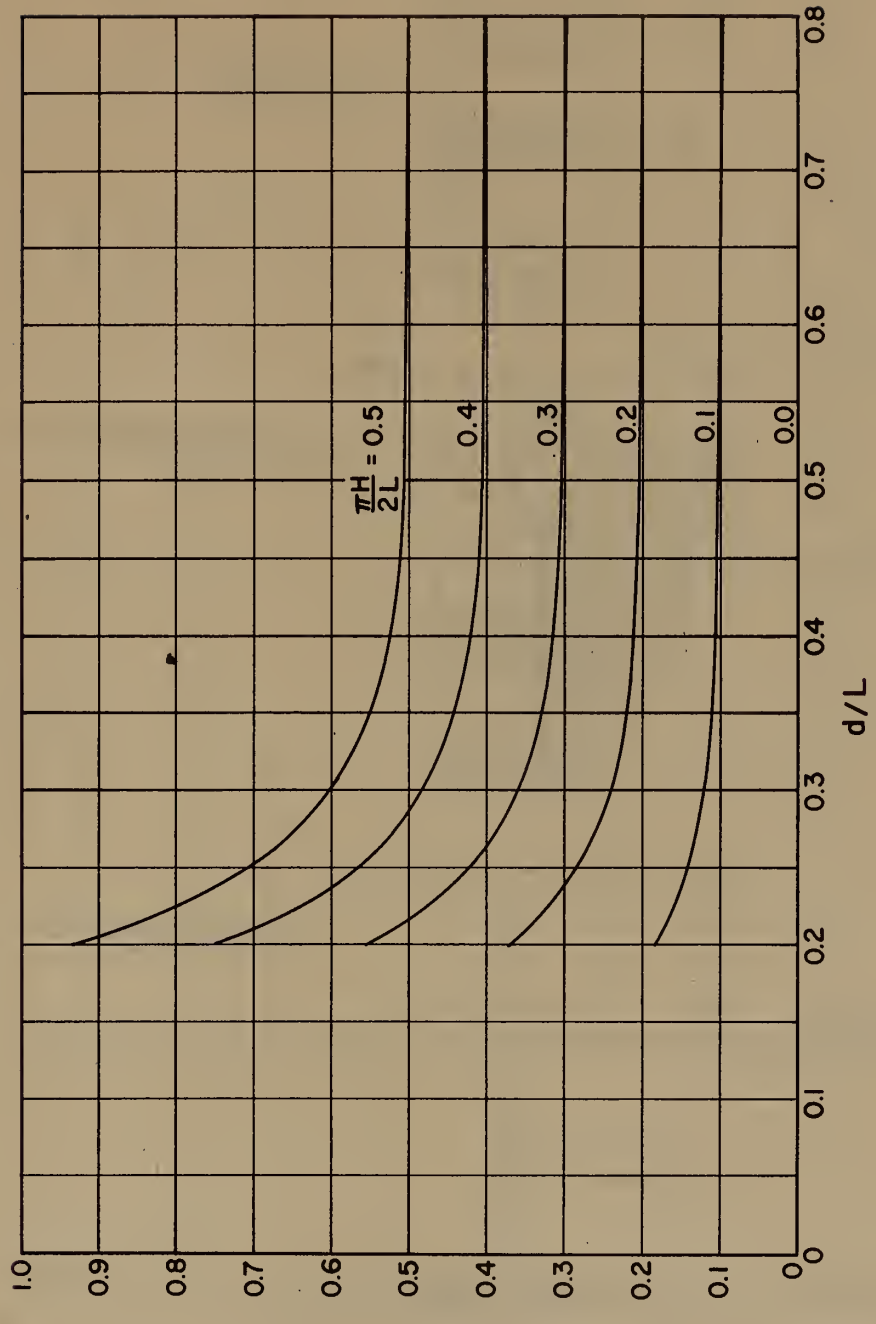
IV D-1: Morison, J.R., H.A. Iverson. "Final Report, Forces on Piling," Series 35 Issue 5, Institute of Engineering Research, University of California, Berkeley, California, Auguast 1951.

TABLE IV D-1

THRUST AND MOMENT ON PILE

| $\frac{2\pi x}{L}$ | $\frac{x}{L}$ | $S_s$ Ft. | $S_s/L$ | $k_1$ | $k_2$ | $k_3$ | $k_4$ | $\theta$ Deg. | $F_H$ Lbs. | $M_{S_1}$ Ft. Lbs. | $\ell_a$ Ft. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | 0 | 43.26 | 0.259 | 0.218 | 1.111 | 0.122 | 0.547 | 0 | 172 | 4,770 | 28 |
| 15 | 0.042 | 43.12 | 0.258 | 0.216 | 1.104 | 0.119 | 0.541 | 18° | 190 | 5,187 | 27 |
| 30 | 0.083 | 42.73 | 0.256 | 0.211 | 1.089 | 0.116 | 0.529 | 35° | 185 | 4,770 | 26 |
| 45 | 0.125 | 42.12 | 0.252 | 0.202 | 1.058 | 0.109 | 0.504 | 52° | 157 | 3,890 | 25 |
| 60 | 0.167 | 41.37 | 0.248 | 0.193 | 1.029 | 0.102 | 0.481 | 67 | 134 | 3,137 | 23 |
| 75 | 0.208 | 40.56 | 0.243 | 0.183 | 0.993 | 0.094 | 0.454 | 83 | 117 | 2,635 | 22 |
| 90 | 0.250 | 39.74 | 0.238 | 0.173 | 0.957 | 0.086 | 0.427 | 97 | 109 | 2,384 | 22 |
| 105 | 0.292 | 39.00 | 0.233 | 0.164 | 0.924 | 0.080 | 0.402 | 112 | 82 | 1,673 | 20 |
| 120 | 0.333 | 38.37 | 0.230 | 0.159 | 0.903 | 0.076 | 0.387 | 126 | 42 | 753 | * |
| 135 | 0.375 | 37.88 | 0.227 | 0.153 | 0.884 | 0.072 | 0.373 | 140 | -5 | -251 | * |
| 150 | 0.417 | 37.53 | 0.225 | 0.150 | 0.871 | 0.070 | 0.364 | 154 | -50 | -1255 | 25 |
| 165 | 0.458 | 37.32 | 0.223 | 0.147 | 0.858 | 0.067 | 0.355 | 167 | -87 | -1966 | 23 |
| 180 | 0.500 | 37.25 | 0.223 | 0.147 | 0.858 | 0.067 | 0.355 | 180 | -116 | -2594 | 22 |
| 195 | 0.542 | 37.32 | 0.223 | 0.147 | 0.858 | 0.067 | 0.355 | 193 | -132 | -3054 | 23 |
| 210 | 0.583 | 37.53 | 0.225 | 0.150 | 0.871 | 0.070 | 0.364 | 206 | -140 | -3096 | 22 |
| 225 | 0.625 | 37.88 | 0.227 | 0.153 | 0.884 | 0.072 | 0.373 | 220 | -137 | -3012 | 22 |
| 240 | 0.666 | 38.37 | 0.230 | 0.159 | 0.903 | 0.076 | 0.387 | 232 | -127 | -2845 | 22 |
| 255 | 0.708 | 39.00 | 0.233 | 0.164 | 0.924 | 0.080 | 0.402 | 246 | -116 | -2594 | 22 |
| 270 | 0.750 | 39.74 | 0.238 | 0.173 | 0.957 | 0.086 | 0.427 | 261 | -113 | -2510 | 22 |
| 285 | 0.791 | 40.56 | 0.243 | 0.183 | 0.993 | 0.094 | 0.454 | 276 | -113 | -2552 | 23 |
| 300 | 0.833 | 41.37 | 0.248 | 0.193 | 1.029 | 0.102 | 0.481 | 291 | -89 | -2092 | 23 |
| 315 | 0.875 | 42.12 | 0.252 | 0.202 | 1.058 | 0.109 | 0.504 | 308 | -45 | -669 | * |
| 330 | 0.916 | 42.73 | 0.256 | 0.211 | 1.089 | 0.116 | 0.529 | 328 | +37 | +1171 | * |
| 345 | 0.958 | 43.12 | 0.258 | 0.216 | 1.104 | 0.119 | 0.541 | 342 | 113 | +3263 | 29 |
| 360 | 1.000 | 43.26 | 0.259 | 0.218 | 1.111 | 0.122 | 0.547 | 360 | 172 | +4770 | 28 |

* Indeterminate




$$\frac{\pi H}{2L}\left[\frac{\cosh\frac{2\pi d}{L}\left(\cosh\frac{4\pi d}{L}+2\right)}{2\left(\sinh\frac{2\pi d}{L}\right)^2}\right]$$

FIG. IV D-1 -- EFFECT OF WAVE STEEPNESS ON SURFACE PROFILE.

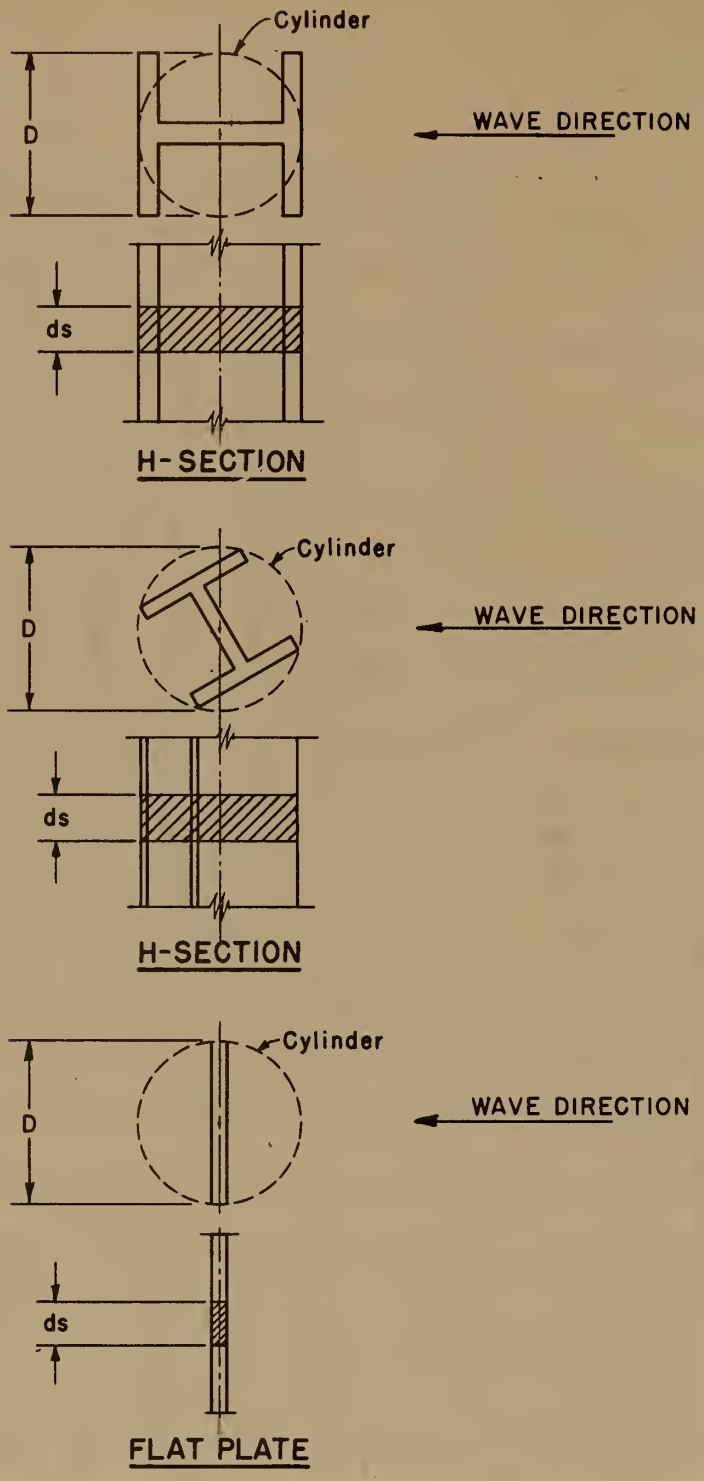



FIG. IV D-2 -- CYLINDRICAL INCREMENT FOR VIRTUAL MASS OF H-SECTION AND FLAT PLATE.

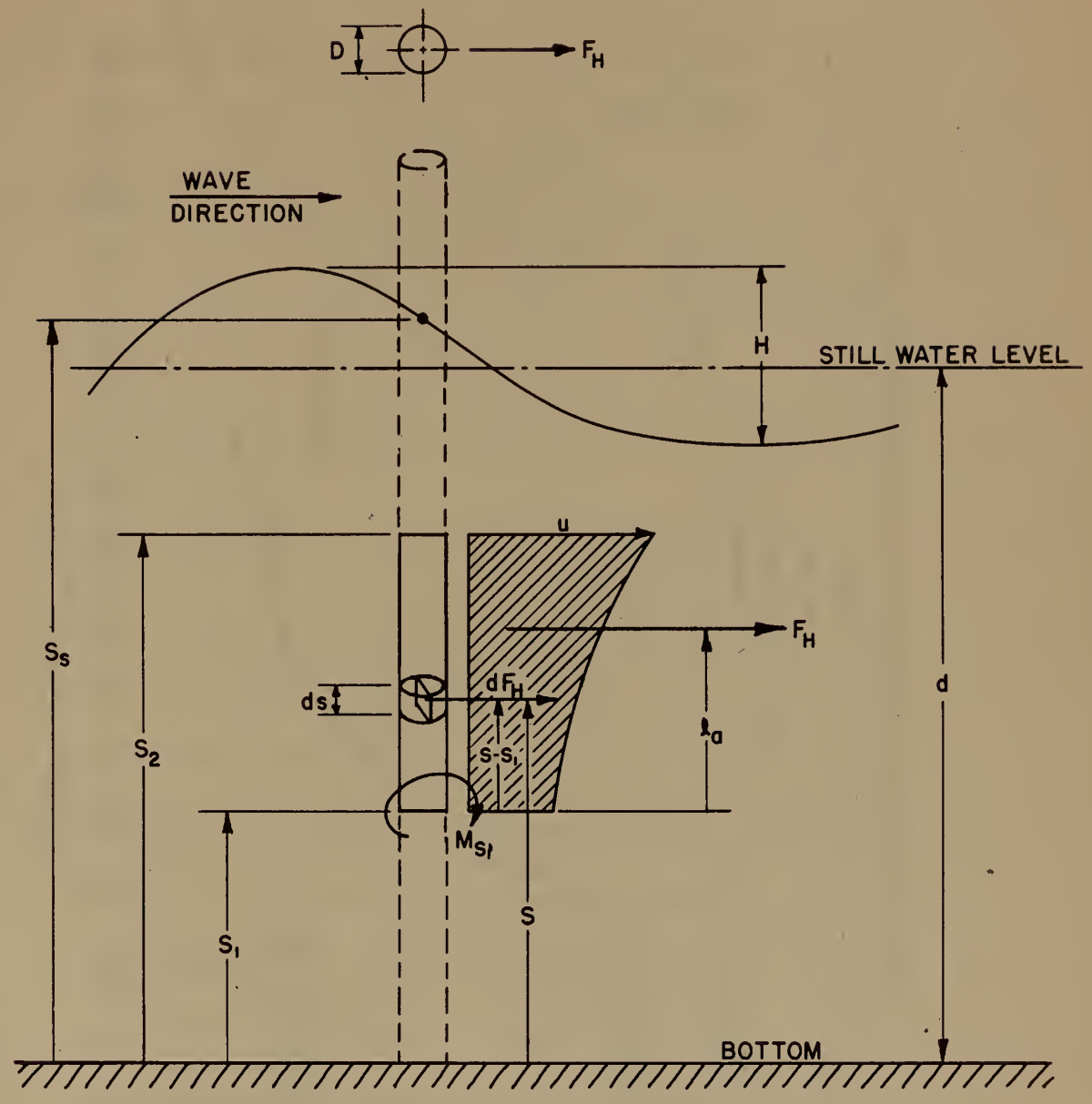



FIG. IV D - 3 -- SECTION OF PILE SHOWING INCREMENT OF FORCE, THRUST, MOMENT, AND POINT OF APPLICATION.

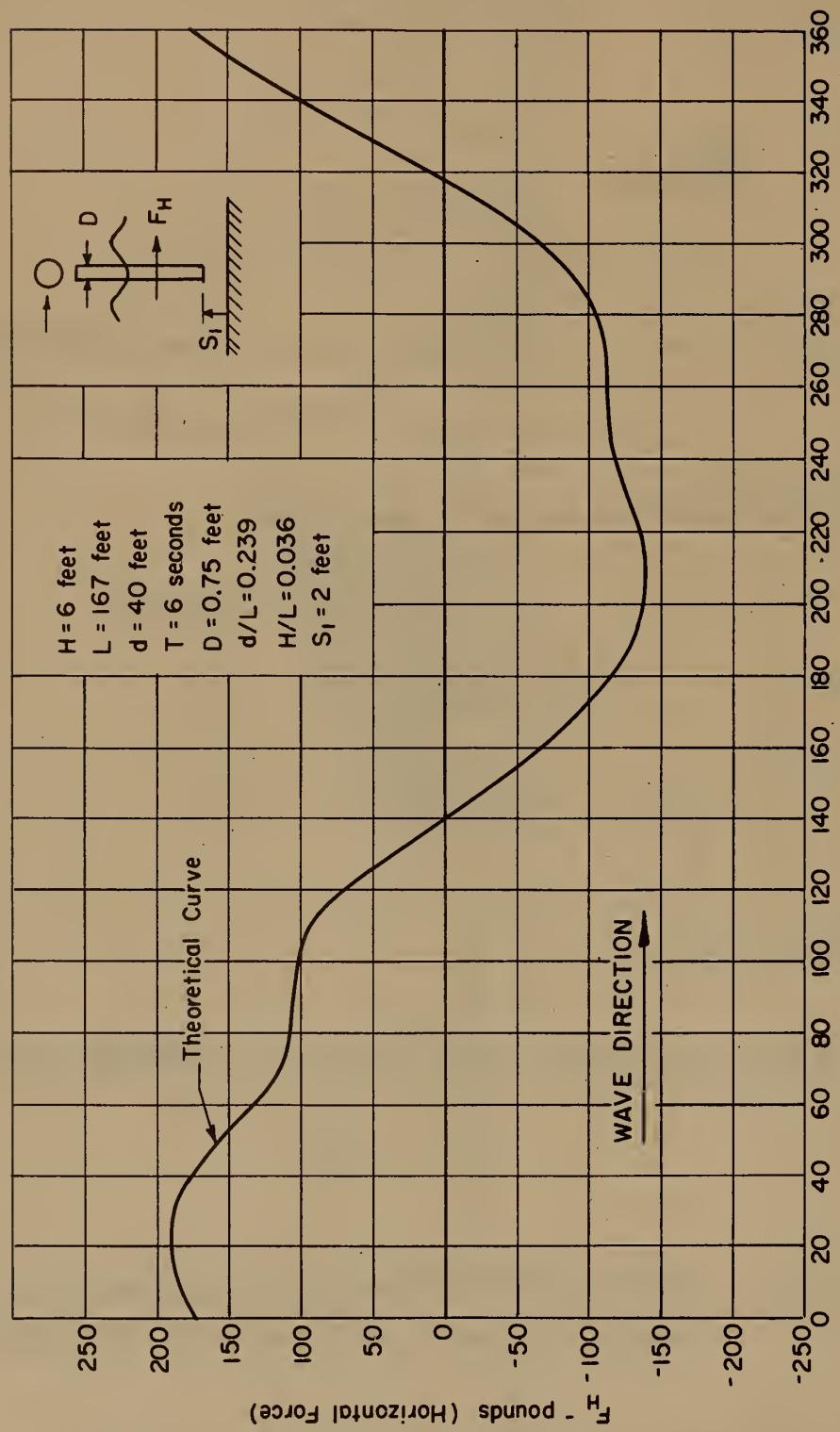



FIG. IXD-4 - HORIZONTAL FORCE VERSUS ANGULAR POSITION

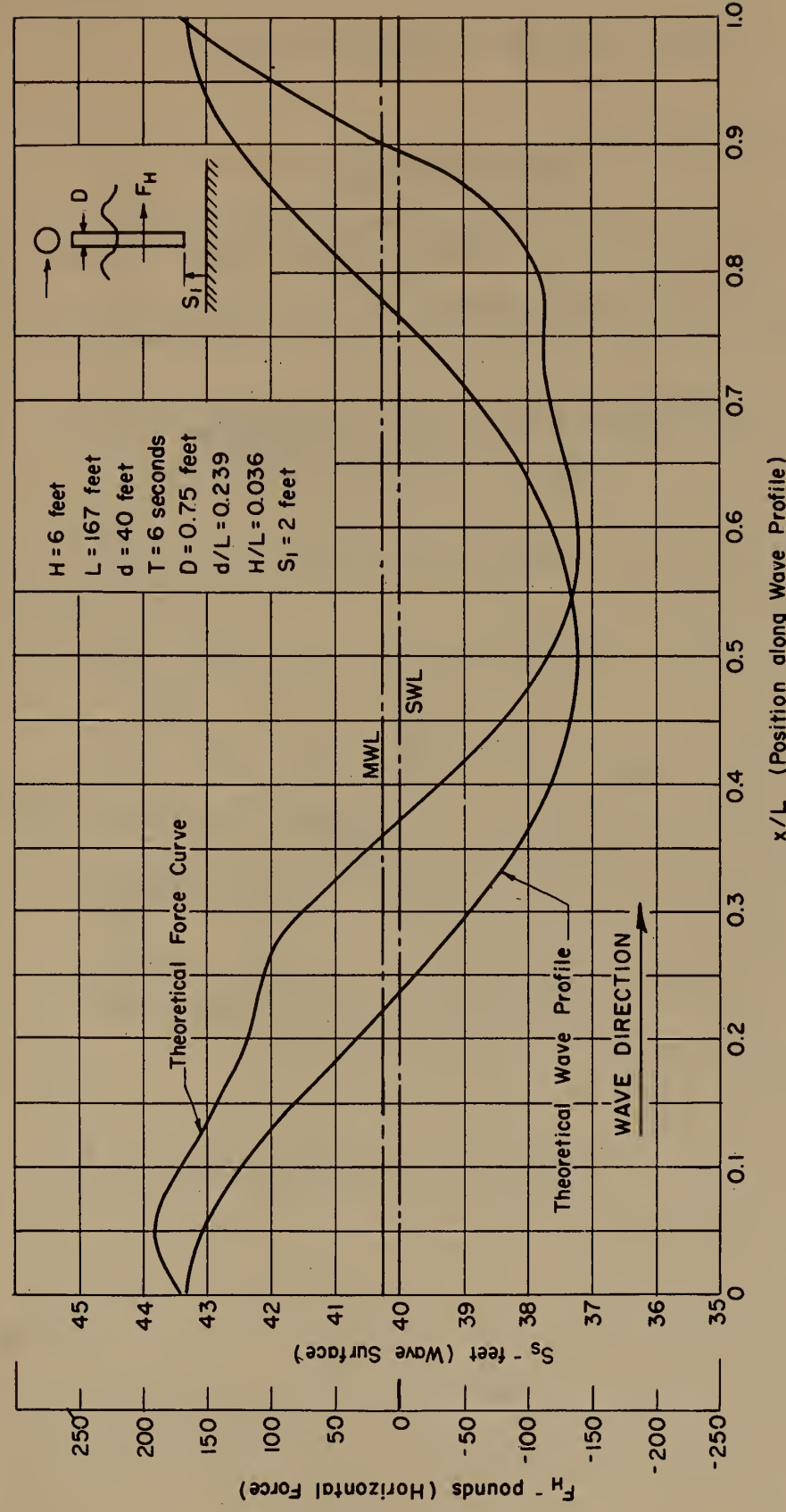



FIG. IV D-5 - TIME HISTORY OF HORIZONTAL FORCE

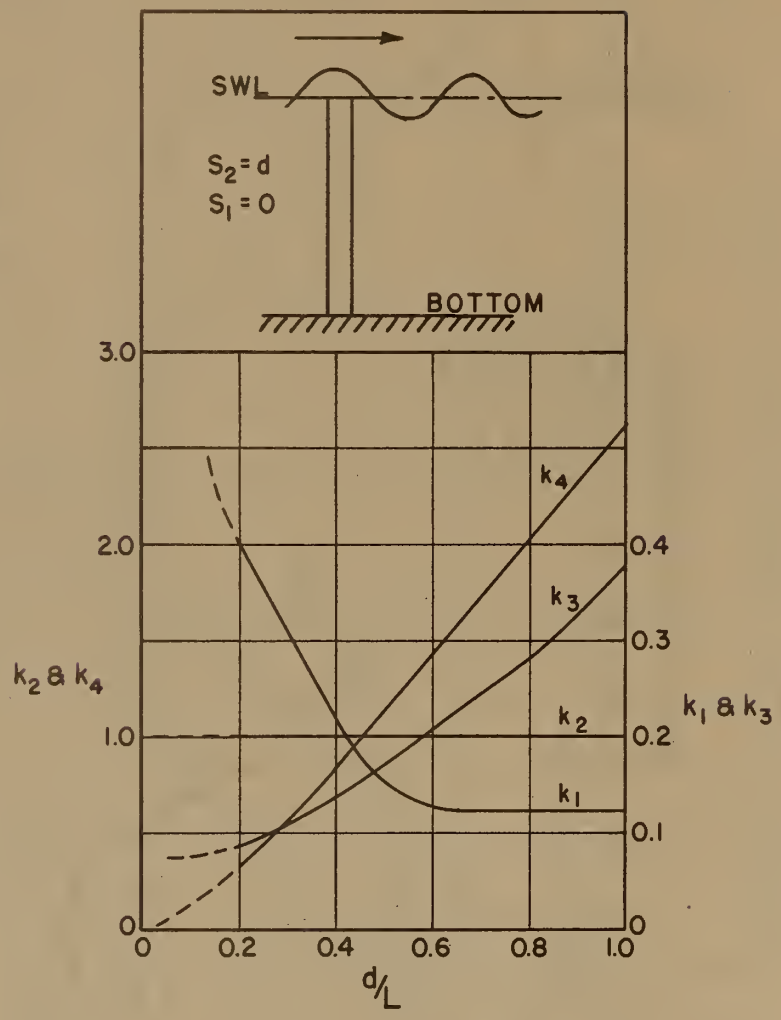



FIG. IV D - 6 -- $k_1$, $k_2$, $k_3$, and $k_4$ vs. $d/L$

MANUAL OF AMPHIBIOUS OCEANOGRAPHY

SECTION IV: WAVE FORCES ON OBJECTS, PILING, VERTICAL WALLS AND RUBBLE MOUNDS

## E: MOMENT EXERTED BY WAVES ON PILING

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## E. MOMENT EXERTED BY WAVES ON PILING

1. Moment at any Point on a Piling - - The total moment about any point on a pile contributed by the wave action above that point is calculated from the increment of force, equation IV D-1.2 and the leverarm integrated over the pile section (Reference IV E-2).

$$M_{S_1} = \int_{S_1}^{S_2} (S - S_1)\, dF_H \qquad \text{(IV E-1.1)}$$

$$= \int_{S_1}^{S_2} (S - S_1)\, \frac{\pi^2}{2} \rho_w C_d D \frac{H^2}{T^2} \frac{\left(\text{Cosh}\frac{2\pi S}{L}\right)^2}{\left(\text{Sinh}\frac{2\pi d}{L}\right)^2} \text{Cos}^2\theta\, dS$$

$$+ \int_{S_1}^{S_2} (S - S_1)\, \frac{\pi^3}{2} \rho_w C_M D^2 H \frac{\text{Cosh}\frac{2\pi S}{L}}{T^2 \text{Sinh}\frac{2\pi d}{L}} \text{Sin}\theta\, dS \qquad \text{(IV E-1.2)}$$

and

$$M_{S_1} = \rho_w \frac{DH^2L^2}{T^2}\left\{C_d k_3 \text{Cos}^2\theta + \frac{\pi D}{4H} C_M k_4 \text{Sin}\theta + \frac{2\pi S_1}{L}\left[C_d \frac{k_1}{2} \text{Cos}^2\theta + \frac{\pi D}{4H} C_M \frac{k_2}{2} \text{Sin}\theta\right]\right\} \qquad \text{(IV E-1.3)}$$

where

$k_1$ = equation IV D-1.6

$k_2$ = equation IV D-1.7

$$k_3 = \frac{\frac{1}{2}\left(\frac{4\pi S_2}{L}\right)^2 - \frac{1}{2}\left(\frac{4\pi S_1}{L}\right)^2 + \frac{4\pi S_2}{L}\text{Sinh}\frac{4\pi S_2}{L} - \frac{4\pi S_1}{L}\text{Sinh}\frac{4\pi S_1}{L} - \text{Cosh}\frac{4\pi S_2}{L} + \text{Cosh}\frac{4\pi S_1}{L}}{64\left(\text{Sinh}\frac{2\pi d}{L}\right)^2} \qquad \text{(IV E-1.4)}$$

$$k_4 = \frac{\frac{2\pi S_2}{L}\text{Sinh}\frac{2\pi S_2}{L} - \frac{2\pi S_1}{L}\text{Sinh}\frac{2\pi S_1}{L} - \text{Cosh}\frac{2\pi S_2}{L} + \text{Cosh}\frac{2\pi S_1}{L}}{2\,\text{Sinh}\frac{2\pi d}{L}} \qquad \text{(IV E-1.5)}$$

If $S_2$ is less than $S_s$ then the position of the maximum moment about $S_1$ is obtained by differentiating equation IV E-1.3, setting in equal to zero, and solving for $\beta_1$,

$$\text{Sin}\beta_1 = \frac{\pi D C_M\left[k_4 - \left(\frac{2\pi S_1}{L}\right)\frac{k_2}{2}\right]}{8 H C_d\left[k_3 - \left(\frac{2\pi S_1}{L}\right)\frac{k_1}{2}\right]} \qquad \text{(IV E-1.6)}$$

where $\beta_1$ is defined as the angular position of the water particle for the maximum total moment about $S_1$. The value $\beta_1$ is a value of $\theta$. The maximum moment is computed by substituting the value of $\text{Sin}\beta_1$ and $\text{Cos}\beta_1$ into equation IV E-1.3.

If $S_2 = S_s$ then an explicit solution for $\beta_1$ is impossible and the position of maximum moment must be found by graphical means. The total moment is computed for the two previous examples.

2. The Total Moment about $S_1$ on a Submerged Section of a Single Round Pile:

Suppose as before that D = 0.75 ft, $S_1$ = 2 ft, $S_2$ = 32 ft, d = 40 ft, and T = 6 sec.

(i) Wave length L = 167 ft. as before (I V C-2a).

$\frac{d}{L} = 0.239$ $\qquad$ $\frac{S_1}{L} = 0.012$

$\frac{H}{L} = 0.036$ $\qquad$ $\frac{S_2}{L} = 0.192$

(ii) The coefficient of drag and mass

$C_d = 1.20$

$C_m = 1.51$ $\qquad$ as before in IV D-2

(iii) Compute the angular position of maximum moment fron equation IV E-1.6

$k_1 = 0.105$ (equation IV D-1.6, as before in IV D-2)
$k_2 = 0.678$ (equation IV D-1.7, as before in IV D-2)

Now compute $k_3$ (equation IV E-1.4) and $k_4$ (equation IV E-1.5)

$$k_3 = \frac{\frac{1}{2}\left(\frac{4\pi S_s}{L}\right)^2 - \frac{1}{2}\left(\frac{4\pi S_1}{L}\right)^2 + \frac{4\pi S_2}{L}\operatorname{Sinh}\frac{4\pi S_2}{L} - \frac{4\pi S_1}{L}\operatorname{Sinh}\frac{4\pi S_1}{L} - \operatorname{Cosh}\frac{4\pi S_2}{L} + \operatorname{Cosh}\frac{4\pi S_1}{L}}{64 \quad \left(\operatorname{Sinh}\frac{2\pi d}{L}\right)^2}$$

$$= \frac{\frac{1}{2}(2.413)^2 - \frac{1}{2}(0.1508)^2 + 2.413 \times 5.538 - 0.1508 \times 0.1513 - 5.625 + 1.0114}{64 \ (2.133)^2}$$

$$= 0.0399$$

$$k_4 = \frac{\frac{2\pi S_2}{L}\operatorname{Sinh}\frac{2\pi S_2}{L} - \frac{2\pi S_1}{L}\operatorname{Sinh}\frac{2\pi S_1}{L} - \operatorname{cosh}\frac{2\pi S_2}{L} + \operatorname{Cosh}\frac{2\pi S_1}{L}}{2 \ \operatorname{Sinh}\frac{2\pi d}{L}}$$

$$= \frac{1.206 \times 1.521 - 0.0754 \times 0.0755 - 1.820 + 1.0028}{2 \ (2.133)}$$

$$= 0.2370$$

$$\operatorname{Sin}\beta_1 = \frac{\pi D C_M}{8 H C_d}\left[\frac{k_4 - \left(\frac{2\pi S_1}{L}\right)\frac{k_2}{2}}{k_3 - \left(\frac{2\pi S_1}{L}\right)\frac{k_1}{2}}\right]$$

$$= \frac{\pi \times 0.75 \times 1.51\left[0.237 - 0.0754 \times \frac{0.678}{2}\right]}{8 \times 6 \times 1.20\left[0.0399 - 0.0754 \times \frac{0.105}{2}\right]} = 0.364$$

$\beta_1 = 21^\circ 21'$ $\qquad$ $\operatorname{Cos}\beta_1 = 0.931$

4. Compute the maximum moment about $S_1$ from equation IV E-1.3

$$M_{S_1} = \rho_w \frac{D H^2 L^2}{T^2}\left(C_d k_3 \operatorname{Cos}^2\theta + \frac{\pi D}{4H} C_M k_4 \operatorname{Sin}\theta - \frac{2\pi S_1}{L}\left[\frac{k_1}{2} C_d \operatorname{Cos}^2\theta + \frac{\pi D}{4H} C_M \frac{k_2}{2} \operatorname{Sin}\theta\right]\right)$$

$$= \frac{2 \times 0.75 \times 6^2 \times 167^2}{6^2} \left\{ 1.20 \times 0.0399 \times 0.931^2 + \frac{\pi \times 0.75}{4 \times 6} 1.51 \times 0.2370 \times 0.364 \right.$$
$$\left. - 0.0754 \left[ \frac{0.105}{2} \times 1.20 \times 0.931^2 + \frac{\pi \times 0.75}{4 \times 6} \times 1.510 \times \frac{0.678}{2} \times 0.364 \right] \right\}$$

$$M_{S_1} = 41,833 \left\{ 0.0416 + 0.0128 - 0.0754 \left[ 0.0547 + 0.0183 \right] \right\}$$
$$= 2,045 \text{ Ft. Lbs.}$$

3. The Total Moment about $S_1$ on a Section of Round Piling that Extends Above the Wave Surface:

Suppose as before that D = 0.75 ft, $S_1$ = 2 ft, d = 40 ft, and T = 6 sec.

(i) The wave length L = 167 ft. as before (IV C-2a).

$$\frac{d}{L} = 0.239 \qquad \frac{S_1}{L} = 0.012$$

$$\frac{H}{L} = 0.036$$

(ii) Compute the values of $S_s$ from Equation IV D-1.1 and then $S_s/L$ as before in IV D-3 (See Table IV D-1)
The coefficients of drag and mass are:

$$\left.\begin{aligned} C_d &= 1.00 \\ C_M &= 1.51 \end{aligned}\right\} \text{as before IV D-3}$$

(iii) Compute the time history of the total moment about $S_1$ from equation IV E-1.3. First we compute $k_1$, $k_2$, $k_3$, and $k_4$ for each $S_s/L$. See Table IV D-1 for the values of $k_1$ and $k_2$.

$$k_3 = \frac{\frac{1}{2}\left(\frac{4\pi S_s}{L}\right)^2 - \frac{1}{2}\left(\frac{4\pi S_1}{L}\right)^2 + \frac{4\pi S_s}{L} \text{Sinh} \frac{4\pi S_s}{L} - \frac{4\pi S_1}{L} \text{Sinh} \frac{4\pi S_1}{L} - \text{Cosh} \frac{4\pi S_s}{L} + \text{Cosh} \frac{4\pi S_1}{L}}{64 \left(\text{Sinh} \frac{2\pi d}{L}\right)^2}$$

for $\frac{2\pi x}{L} = 0^\circ$

$$k_3 = \frac{\frac{1}{2}(3.255)^2 - \frac{1}{2}(0.1508)^2 + 3.255(12.94) - 0.508(0.1513) - 12.98 + 1.0114}{64(2.133)^2}$$

$$= 0.186$$

and

$$k_4 = \frac{\frac{2\pi S_s}{L}\,\mathrm{Sinh}\,\frac{2\pi S_s}{L} - \frac{2\pi S_1}{L}\,\mathrm{Sinh}\,\frac{2\pi S_1}{L} - \mathrm{Cosh}\,\frac{2\pi S_s}{L} + \mathrm{Cosh}\,\frac{2\pi S_1}{L}}{2\,\mathrm{Sinh}\,\frac{2\pi d}{L}}$$

for $\frac{2\pi x}{L} = 0^\circ$

$$k_4 = \frac{1.627\,(2.477) - 0.0754(0.0755) - 2.643 + 1.0028}{2(2.133)} = 0.547$$

etc. See Table IV D-1.

Then each set of values of $k_1$, $k_2$, $k_3$, and $k_4$ and each $\theta$ is substituted into the moment equation

$$M_{S1} = \rho_w \frac{D H^2 L^2}{T^2}\left\{\pm C_d\, k_3 \cos^2\theta + \frac{\pi D}{4H} C_M k_4 \sin\theta - \frac{2\pi S_1}{L}\left[C_d \frac{k_1}{2}\cos^2\theta + \frac{\pi D}{4H} C_M \frac{k_2}{2}\sin\theta\right]\right\}$$

$$= \frac{2 \times 0.75 \times 6^2 \times 167^2}{6^2}\left\{\pm 1.00\, k_3 \cos^2\theta + \frac{\pi \times 0.75}{4 \times 6} 1.51\, k_4 \sin\theta - 0.0754\left[1.00 \frac{k_1}{2}\cos^2\theta + \frac{\pi \times 0.75}{4 \times 6} 1.51 \frac{k_2}{2}\sin\theta\right]\right\}$$

for $\theta = 0^\circ$

$$M_{S_1} = 41,833\left\{1.00 \times 0.122 \times 1^2 + 0 - 0.0754\left[1.00 \times \frac{0.218}{2}(1)^2 + 0\right]\right\}$$

$$= 4,770 \text{ Ft. Lbs.}$$

etc. See Table IV D-1. Now the values of $M_{S_1}$ are plotted versus $\theta$ to determine the position and amplitude of the maximum moment about $S_1$, see Figure IV E-1.

Maximum $M_{S_1} = 5,187$ Ft. Lbs.

and $\beta = 18^\circ$

The time history of the total moment along the wave profile is shown in Figure IV E-2.

4. Discussion of Total Moment -- The time history of the total moment about the bottom for a round pile, an H-section, and a flat plate (Reference IV E-2 and 3) are shown in Figures IV E-3, 4 and 5 (where $M_T$ is the total moment amplitude). The measured time history of moments in these examples were obtained from model studies. One effect of the H-section and flat plate over the round pile is an increase in the coefficient of drag. In the particular case of the H-section, the orientation effects the force and the coefficient of drag. The maximum force occurs when the flow was 45 degrees to the axis of the web as shown in Figure IV E-6. This effect of orientation was determined for oscillatory flow and confirmed by a steady flow model study. (Reference IV E-3).

The effect of varying pile size, the wave characteristic and the water depth on the maximum total moment about $S_1$ and the corresponding angular position are found by examining equation IV E-1.3 and 1.6. If the pile diameter is increased the moment is increased and the inertia component becomes increasingly important.

The angle $\beta_1$ increases with an increase in the pile size. If the wave height is increased, the moment is increased and the drag component becomes more important. The angle $\beta_1$ decreases with an increase in the wave height. An increase of the wave period causes an increase in the wave length and thus a decrease of the value of d/L. The increase of the period causes an increase in the moment. The decrease of the values of d/L causes a decrease in the values of $k_3$ and $k_4$ as shown in Figure IV D-6. Since $k_4$ decreases more rapidly than $k_3$ the angle $\beta_1$ decreases with a decrease of d/L, that is for increasing wave periods the drag component becomes more important for the calculation of the total moment. A decrease in the depth of water causes a decrease in the total moment and the same effect as previously discussed for decreasing values of d/L. The coefficient of drag and mass shown in Figure IV C-4 and Tables IV C-1 and 2 are taken as constants over a large range of wave conditions so long as there is no appreciable change of Reynolds numbers. In addition, the parameter $B^2d/H^2L$ can be used to describe the angular position $\beta_1$ as shown in Figure IV E-7. This result was obtained from model studies (Reference IV E-1). Figure IV E-8 shows the relationship of $C_M$ and $C_d$ to $\beta_1$ for the same model studies. Notice that $C_d$ remains relatively constant and that $C_M$ is somewhat more variable. However, there is not enough information about the coefficient of mass, $C_M$, to treat it as a variable in the equation of force and moment.

5. The Point of Application:

The point of application of the total horizontal thrust is the distance above the bottom of the pile section so that when multiplied by the total thrust it will give the total moment about the bottom of the pile section. For the section of a piling discussed previously in IV D-2 and IV E-2 the point of application is given by the equation

$$\ell_a = \frac{M_{S_1}}{F_H} \qquad \text{(IV E-5-1)}$$

For the example of the pile that extends above the wave surface, (IV D-3 and IV E-3), the point of application for the maximum moment at $\beta_1 = 18^\circ$ is

$$\ell_a = \frac{5,187}{190} = 27 \text{ ft.}$$

or since $S_1 = 2$ ft. the total horizontal force would be applied 29 ft. above the bottom where the surface of the water is 43.75 ft. above the bottom. In this example the time history of $\ell_a$ was computed and found to vary between 20 and 29 ft. above $S_1 = 2$ ft. as shown in Table IV D-1.

6. References:

IV E-1: Morison, J. R. "Design of Piling". Paper, Institute of Coastal Engineering; Long Beach, California; October 13, 1950.

IV E-2: Morison, J.R., H.A.Iverson. "Final Report, Forces on Piling", Ser.35, Issue 5 Institute of Engineering Research, University of California, Berkeley California, August 1951.

IV E-3: Morison, J. R. "The Forces Exerted by Waves on Marine Structures". Series 35, Issue 3, Institute of Engineering Research, University of California; October 20, 1950.

IV E-4: Munk, W. H. "Wave Action on Structures". Petroleum Technology, AIME; March 1948.

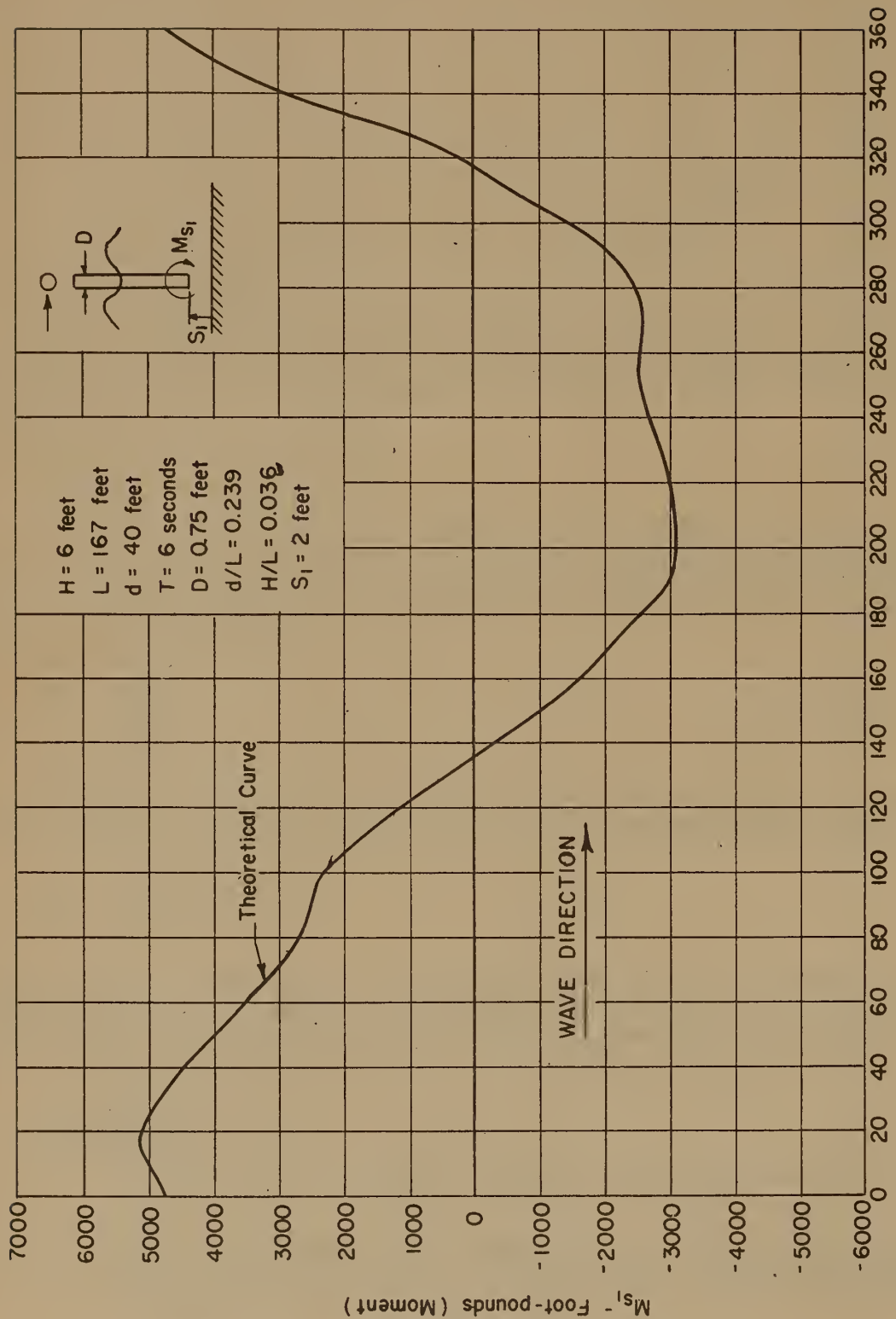



FIG. IV E-1 - MOMENT VERSUS ANGULAR POSITION

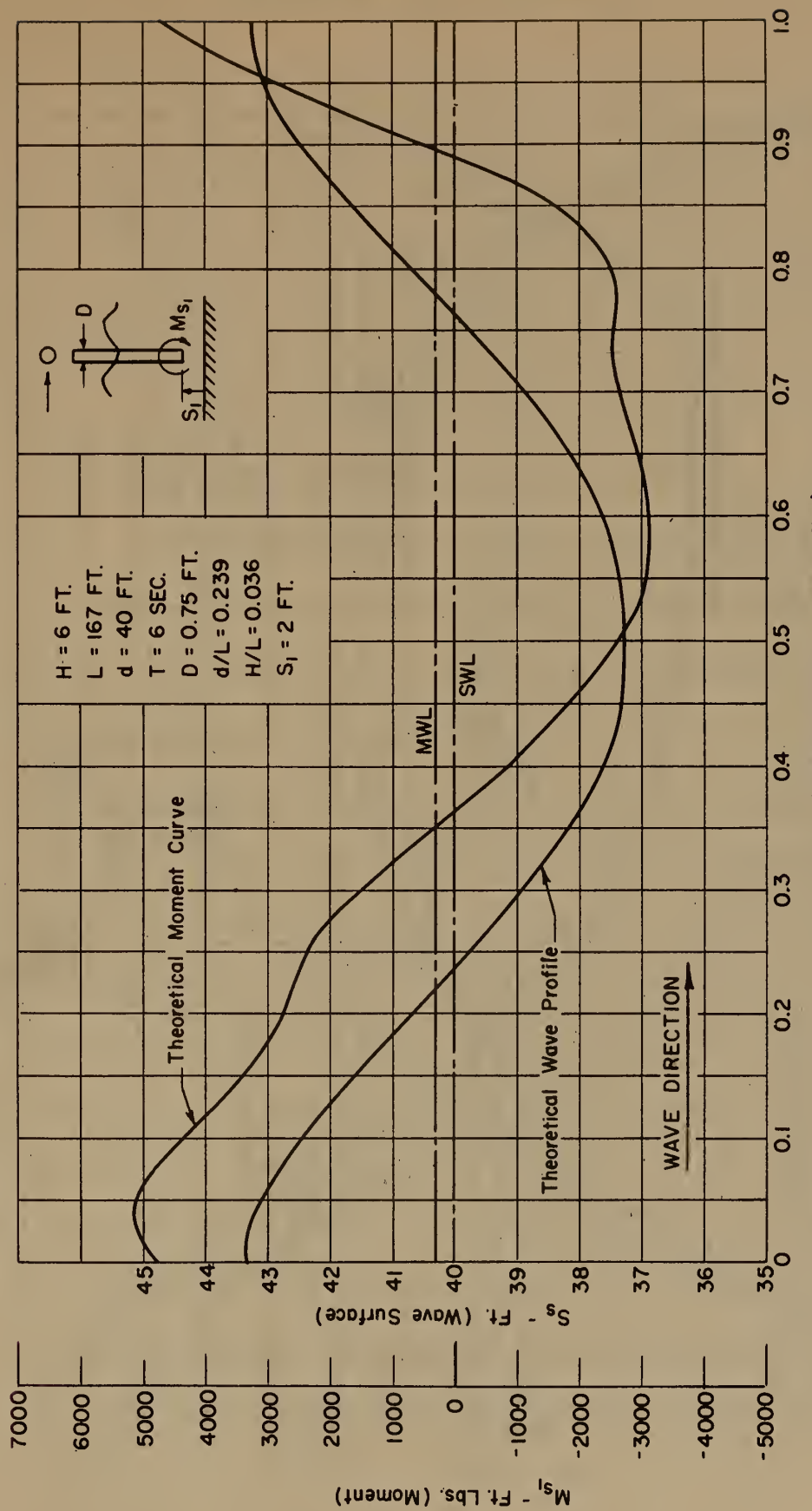



FIG. IXE-2 - TIME HISTORY OF MOMENT ABOUT $S_1$

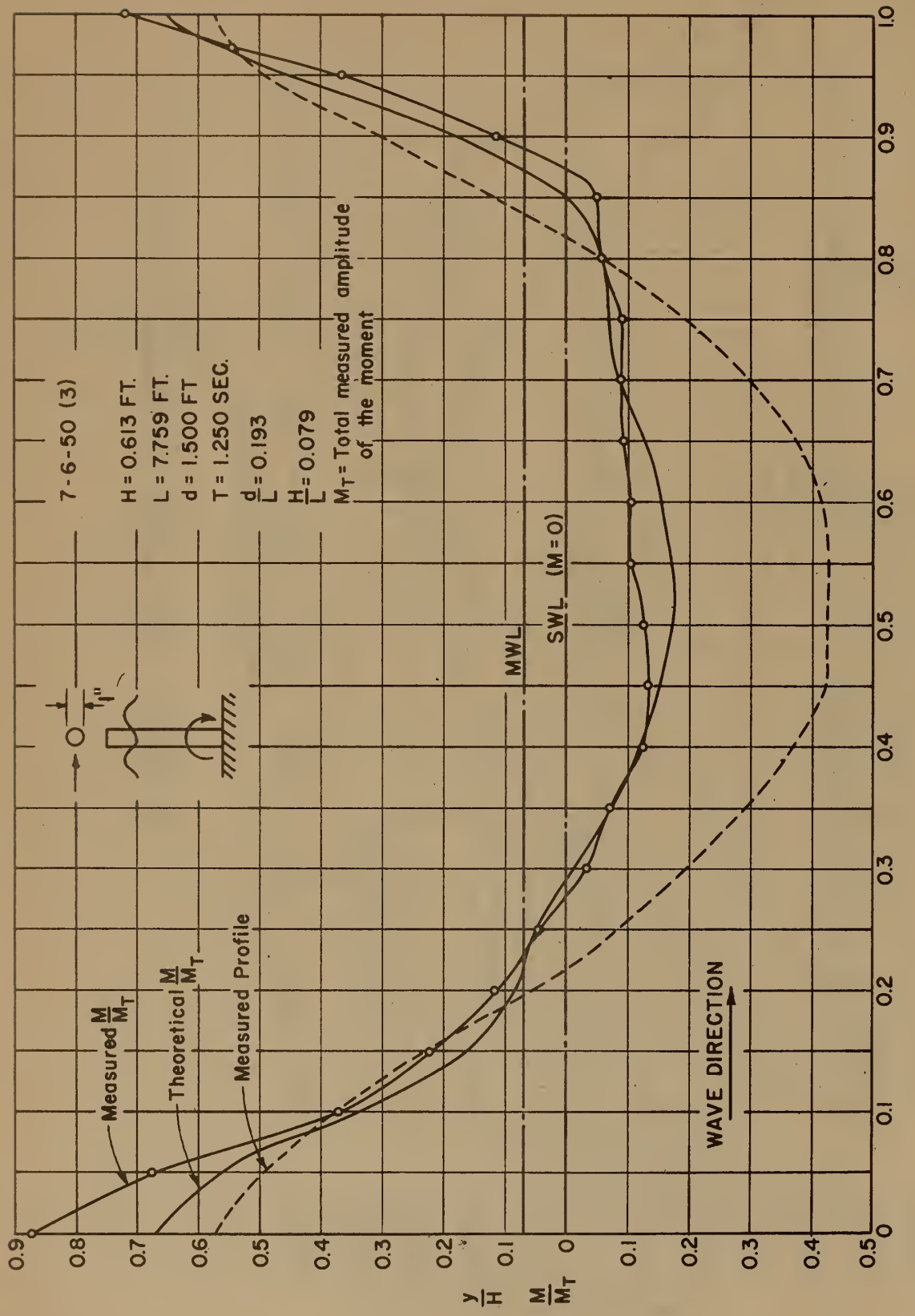



FIG. IV E-3- TIME HISTORY OF TOTAL MOMENT FOR ROUND PILE.

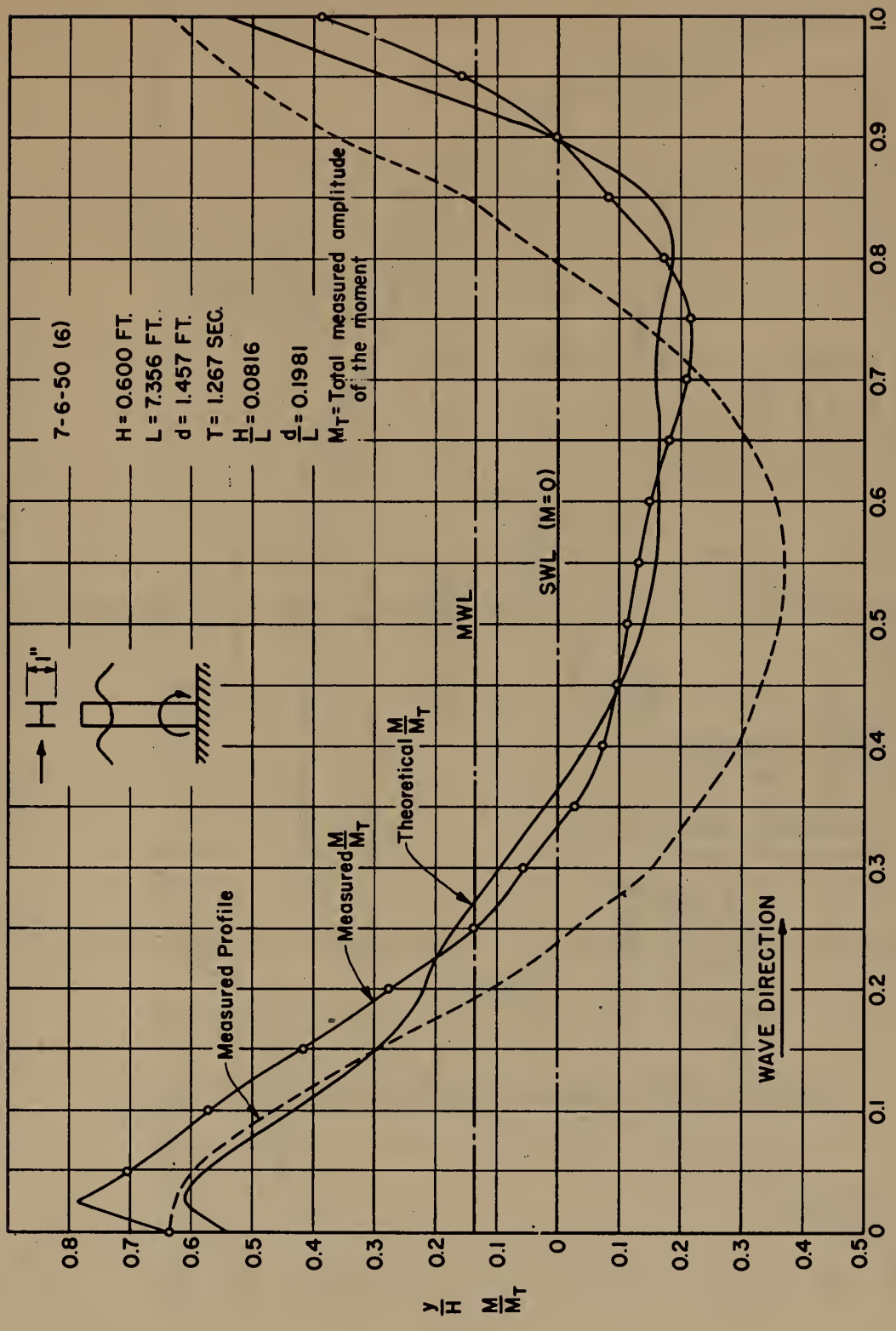



FIG. IX E-4 - TIME HISTORY OF TOTAL MOMENT ON H-SECTION.

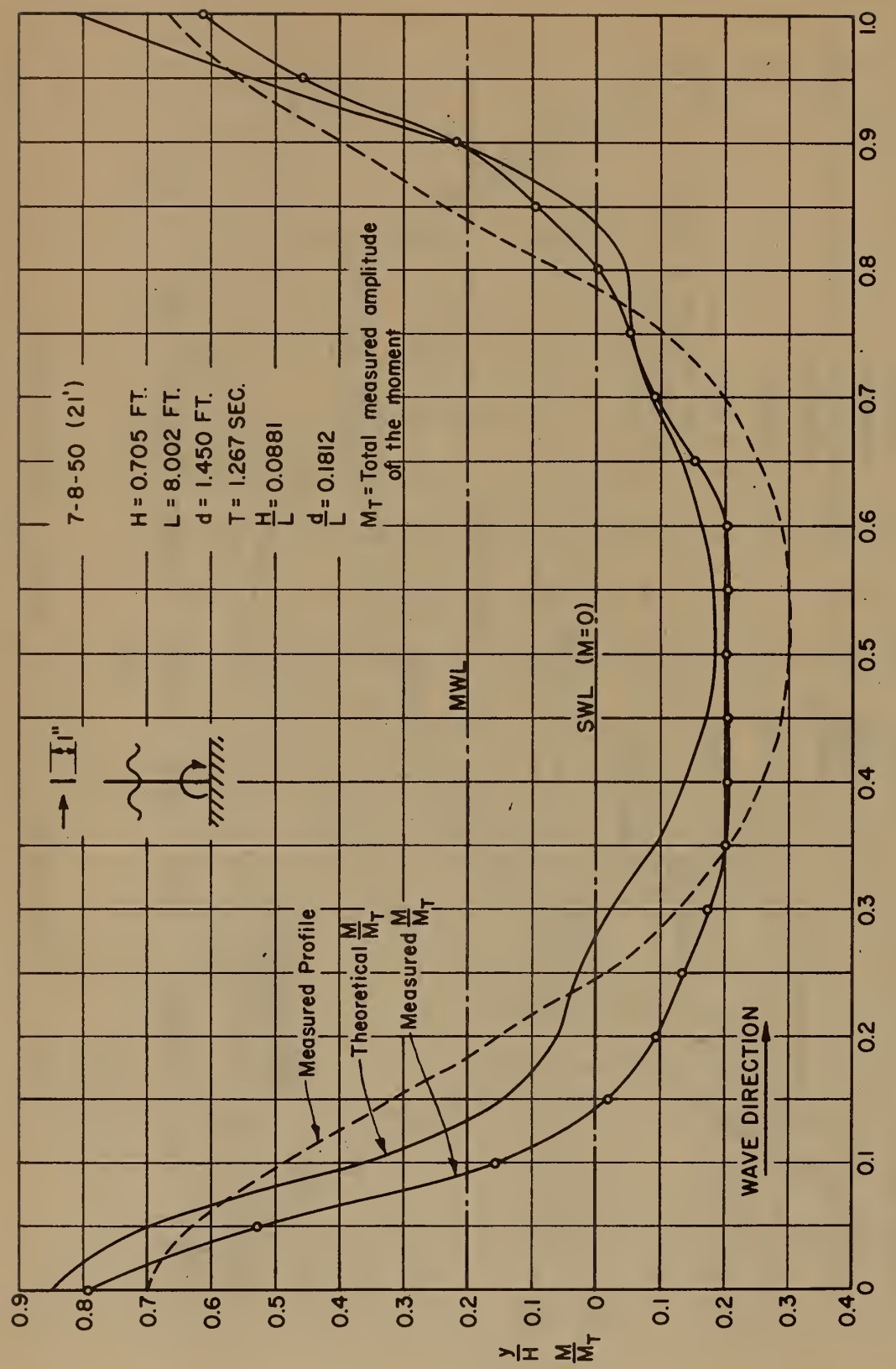



FIG. IX-5 - TIME HISTORY OF TOTAL MOMENT ON FLAT PLATE.

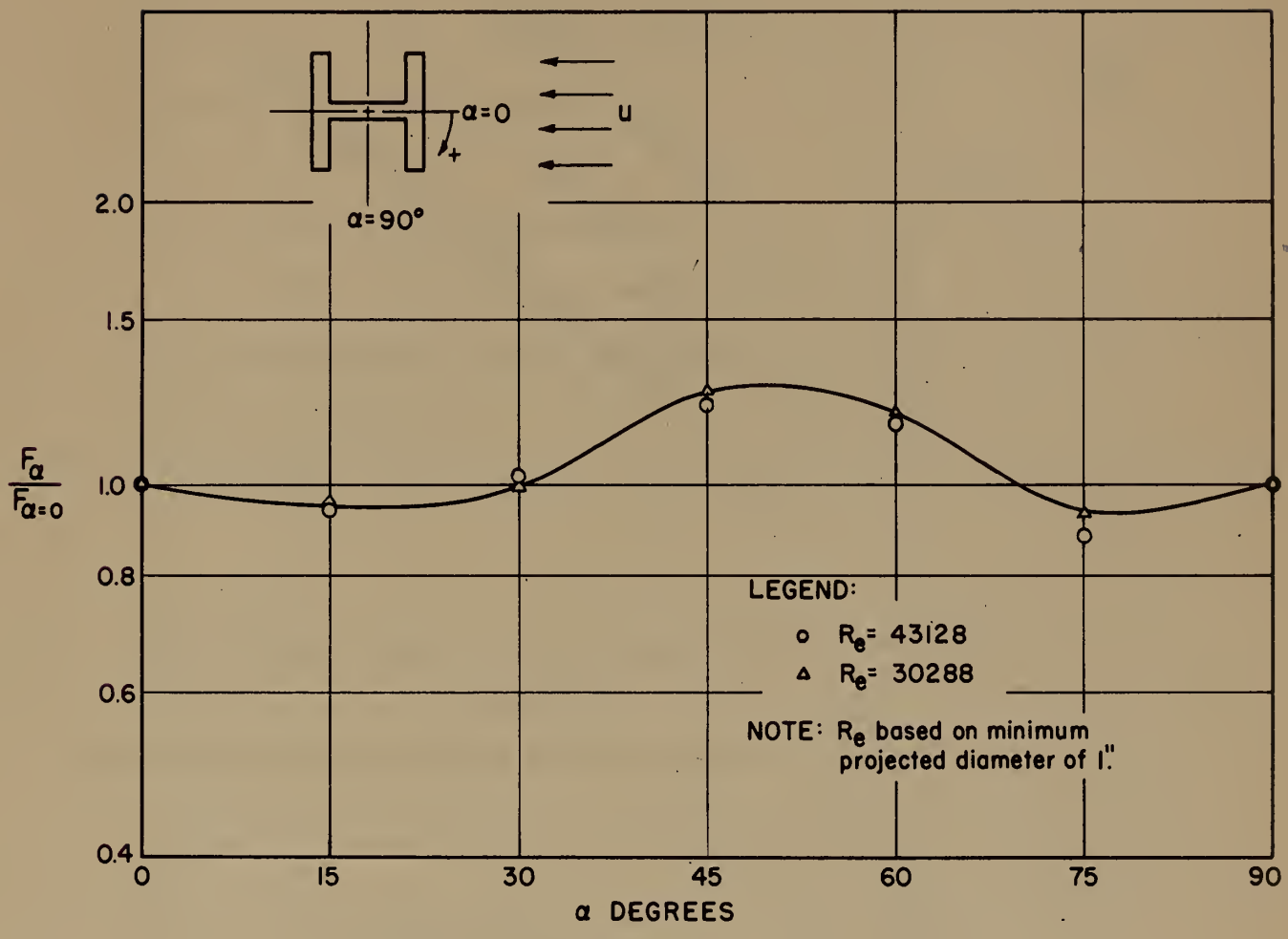



FIG. IV E-6 -- UNIFORM FLOW ABOUT AN H-SECTION.

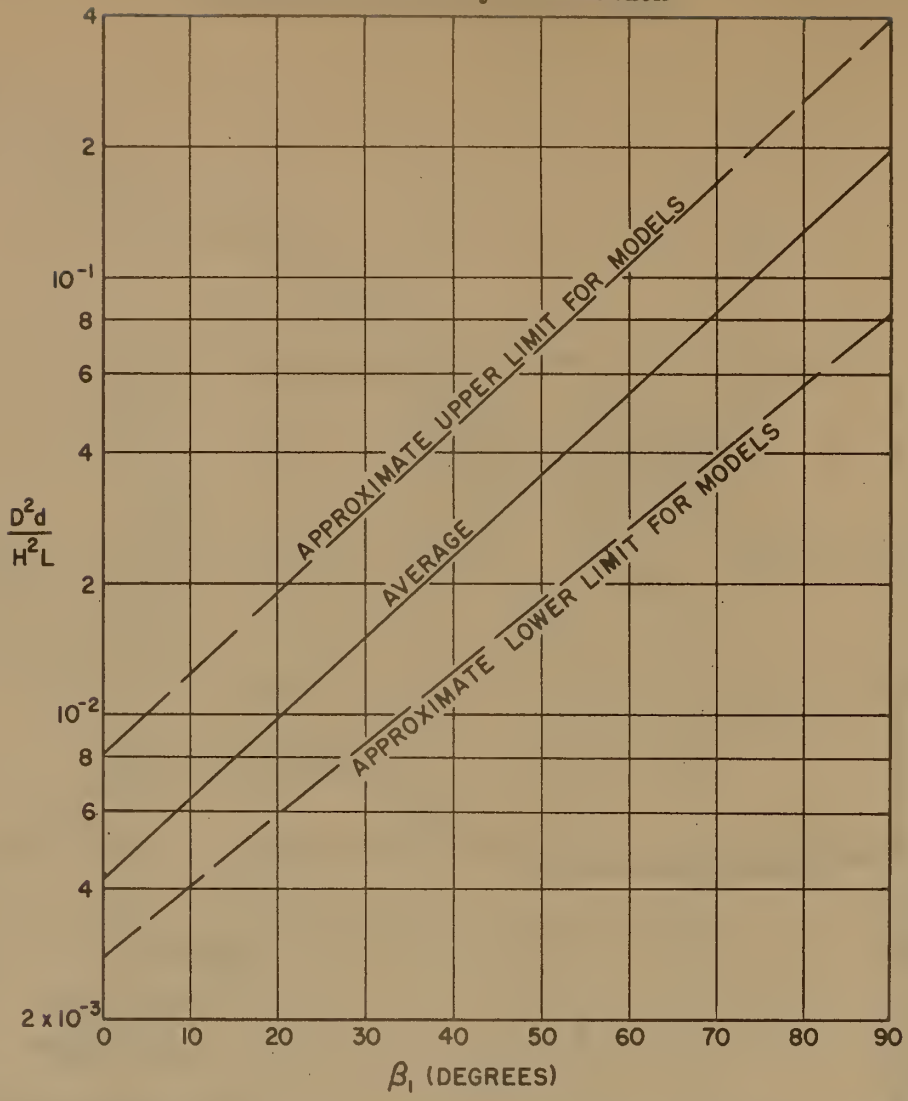



FIG. IV E-7 -- PARAMETER DESCRIBING ANGULAR POSITION.

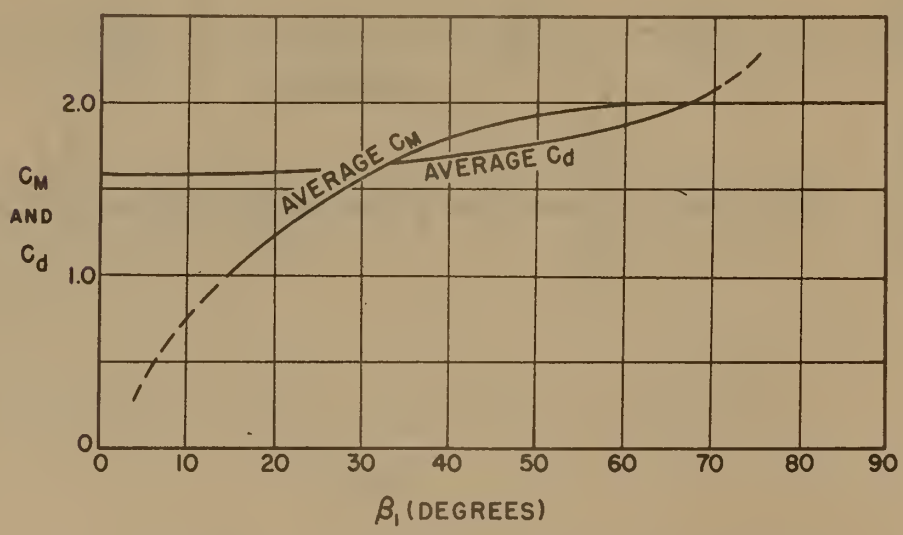



FIG. IV E-8 -- RELATIONSHIP OF $\beta_1$ TO $C_d$ AND $C_M$

# SECTION IV: WAVE FORCES ON OBJECTS, PILING, VERTICAL WALLS AND RUBBLE MOUNDS

## F: EXPERIMENTAL INVESTIGATIONS ON PILING

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## F. EXPERIMENTAL INVESTIGATIONS ON PILING

### 1. Laboratory Investigations:

#### a. Uniform Diameter Piling:

Laboratory measurements were planned on the basis of equation IV E-1.3 and were conducted in the one foot wave channel in the Fluid Mechanics Laboratory at the University of California. In the first series of tests model piles were constructed with a hinge which could be located at any point throughout the depth of the water. The necessary restraining force was measured at a known moment arm above the hinge. The wave profile at the pile and the restraining force on the pile were recorded simultaneously on the same record. By moving the hinge point from point to point throughout the water depth the moment distribution was determined for a given wave condition. Experiments were made with three different pile diameters ($\frac{1}{2}$", 1", 2") which were subjected to a moderately steep wave (H/L = 0.0370) in shallow water. The test conditions are summarized in Table IV F-1 (Reference IV F-2, 3, and 5)

For comparison with the experimental data equation IV E-1.3 was used to calculate the theoretical moment distributions. This comparison between theoretical and experimental values for the three pilings that were tested is shown in Figure IV F-1. A dimensionless plot showing the ratio of the moment of any elevation y to the maximum moment (at the bottom) as a function of relative depth y/d is shown in Figure IV F-2. This figure shows that within the limits of the experiments the moment distribution is independent of pile diameter. The coefficients of mass and drag and the phase angle were different for each pile and were calculated from the measured time history of the total moment about the bottom of the pile.

The shape of the moment distribution curves, Figures IV F-1 and 2, is typical for the exponential force distribution exerted by wave action on a pile. To obtain the force distribution from the moment distribution, the slope of the shear curve must be determined. The shear curve is determined by finding the slope of the moment distribution curves. Theoretically this is possible and can be done for the portion of the pile below the elevation of about the wave trough. For the portion of the pile above the trough elevation practical considerations make it difficult to estimate accurately the shear diagram from the moment curve since near the water surface the moment diagram goes to zero tangentially, indicating the fact that the force approaches a maximum, and the lever arm approaches zero.

#### b. Non-uniform Diameter Piling:

In addition to the tests on piles of uniform diameter an investigation was made on the moment distribution on the model pile of variable diameters shown in Figure IV F-3 (Reference IV F-4). This pile was constructed such that the moment at any point (designated hinge point in Figure IV D-3) resulting from wave forces above the point could be computed from a measured restraining force acting on a known moment arm. The moment distribution was measured for this pile when acted upon by a relatively steep wave (H/L = 0.0971).

The theoretical moment distribution on the variable diameter pile was computed by modifying equation IV F-1.3 to the form:

$$M_n = \sum_{j,i} \frac{\rho_w D_j H^2 L^2}{T^2} \left\{ C_d\, k_{3i}\, \cos^2\theta + \frac{\pi D_j}{4H}\, C_M\, k_{4i}\, \sin\theta - \frac{2\pi S_n}{L} \left[ C_d \frac{k_{1i}}{2} \cos^2\theta + \frac{\pi D_j}{4H} \frac{C_M k_{2i}}{2} \sin\theta \right] \right\} \qquad \text{(IV F-1.1)}$$

where

n = 2, 3....11 is the elevation at which total moment is being calculated.
j = 1, 2, 3, 4, and 5 represents the changes in the diameter of the pile (Figure IV F-3) for various i.
i = 2, 3,...n describes the elevation above the bottom under consideration. The elevation for (i) is the same as for (n) and the following relationship of j to i is necessary.

| | |
|---|---|
| j = 1 | i = 2,3 |
| j = 2 | i = 4,5 |
| j = 3 | i = 6,7 |
| j = 4 | i = 8,9 |
| j = 5 | i = 10,11 |

Hence for any n, the summation is carried out for successive values of j and the corresponding values of i until i = n. The expressions for $k_{1i}$, $k_{2i}$, $k_{3i}$ and $k_{4i}$ are the same as for $k_1$, $k_2$, $k_3$ and $k_4$, that is equation IV D-1.6, IV D-1.7, IV E-1.4 and IV E-1.5 where $S_2$ becomes $S_{i-1}$ and $S_1$ becomes $S_i$.

The moment distribution computed by equation IV F-1.1 is compared with the measured moment distribution in the dimensionless plot shown in Figure IV F-4. In the calculations for the theoretical distribution the overall coefficients of drag and mass were obtained from Table IV C-1 and IV C-2 and the phase angle was obtained from the experimental record. The agreement between the theoretical and experimental moment distributions shown in Figure IV F-4 can be considered to be within the limits of experimental accuracy.

c. Effect of Size, Shape and Spacing of Piling -- In order to determine the effect of size, shape, and spacing of piling on the magnitude of the total moment, a series of tests were made in the wave channel on various types of model piling. The first series of tests consisted of single piles (round, flat plates, and an H-section with various orientations) of the dimensions shown in Figure IV F-5. Each pile was hinged at the bottom in the wave channel (with a uniform depth) and subject to waves of various steepnesses. In these tests the total moment at the bottom of the pile and the wave characteristics were measured simultaneously. Summarized in Table IV F-2 are the results of these tests in which the maximum moment on the various piling are compared to the maximum moment on the 1 inch round pile. (Reference IV F-2 and 5). This comparison shows that for unbroken waves the greatest moment occurs on an H-section when the web is oriented at 45 degrees to the direction of wave travel. That orientation of an H-section which might be expected to cause a maximum moment is shown by Figure IV E-6, wherein the ratio of forces is plotted against orientation for two different steady-state conditions. These steady-state tests were made in a wind tunnel on the same model pile as used in the wave channel studies. The data in Table IV F-2 indicates that when a wave has broken, the orientation of an H piling has little significance, primarily be - cause of the variability of the breakers themselves.

In addition to the measurement of moment on single piling, rows of one-inch circular piles and one-inch flat plates were arranged as shown in Figures IV F-6a-b.

For each spacing the piling was subject to the action of three different wave conditions. In each test the maximum moment on the center member was measured. The results of these tests are summarized in Table IV F-3 where the ratio of the maximum moment on the center pile in a row to the maximum moment previously observed on a single one-inch diameter pile is given for waves of various steepnesses acting on circular piles and flat plates. These data for piling perpendicular to wave travel, indicate that when the gap exceeds $1\frac{1}{2}$ D the effect of mutual interference is negligible. This critical spacing probably depends somewhat on the ratio of wave height to pile diameter, which in these tests only varied from H/D = 4.5 to 8.2.

In addition to the tests on a row of piling arranged perpendicular to the direction of wave travel, a limited series of tests were made to determine the effect of mutual interference when a row of piling was arranged parallel to the direction of the waves as shown in Figure IV F-6c. These tests were made on a row of three one-inch diameter piles in which the total moment was measured on the center or protected pile. The results of these tests are shown in Table IV F-3, where again the ratio of the maximum moment on the center pile of the row to the maximum moment of a single pile is shown to be related to the pile spacing. Within the limits of the maximum gap used (a = $1\frac{1}{2}$D) these data show that the center pile was receiving appreciable protection from the adjacent piles.

d. Discussion of Forces on Piling:

In the case of the non-uniform piling, the moment equation IV F-1.1 became more complicated and tedious to evaluate. It is possible to compute the force and moment on any piling where the diameter changes with depth by a known relationship. However, this type of calculation is complicated and tedious and might have to be done graphically. Although difficult this type calculation can be made.

2. Field Investigation:

Forces Exerted by Breaking Waves on Piling:

Many pile structures are subject to the forces of breaking waves. This type of wave force is mostly impact, but not entirely, with the impact occurring as the wave crest strikes the pile. The magnitude of the force is 5 or more times the force of an unbroken wave (Reference IV F-6). Experiments have been made both for model and prototype to determine the character and magnitude of this type of impact loading. No definite conclusions have been obtained. There is no definite method of calculating this impact force.

At Monterey, California a $3\frac{1}{2}$ inch brass pile was placed in the surf zone and the total moments exerted by swell, peaked-up waves, breaking waves and foam lines were measured. Due to the changing tide stage, a theoretical dedinition was made in that the total moment was reduced to a resultant force acting midway between the elevation of the crest and the bottom at the theoretical point of application. The results of this field test are shown in Figure IV F-7 where it can be observed that the foam line exerted the highest force. (Reference IV F-1 and 7) The observations were made on a beach of slope 1 to 18. The breakers were the plunging type. The average coefficient of drag was 0.7 for the swell. The resultant forces was found to be a function of the water depth and the wave height. The total moment measured was proportional to the wave height squared, the wave velocity squared and the elevation of the crest squared divided by still water depth squared. The point of application and the depending upon variables of the resultant force and total moment about the bottom can be obtained from the theory presented in this section parts IV D and IV E by reducing the equation to very shallow water

($\frac{d}{L} \leq 0.027$). This is done by replacing the Sinh $\frac{2\pi x}{L}$ with $\frac{2\pi x}{L}$ and so forth. For the small diameter pile in very shallow water the inertia force is neglected compared to the drag force (Reference IV F-1). Figure IV F-8 shows the comparison of a dimensionless force ratio to the wave height for the $3\frac{1}{2}$" diameter field pile and the $\frac{1}{2}$", 1" and 2" diameter model piles. The comparison of theoretical work, model studies and field studies is fairly good but more prototype information of types of wave conditions is needed.

3. References:

IV F-1: Morison, J.R., H.A.Iverson. "Final Report, Forces on Piling", Series 35, Issue 5, Institute of Engineering Research University of California, Berkeley California, August 1951.

IV F-2: Morison, J. R. "The Forces Exerted by Waves on Marine Structures". Series 35, Issue 3, Institute of Engineering Research, University of California, Berkeley, California, October 20, 1950.

IV F-3: Morison, J. R. "Moment Distribution Exerted by Waves on Piling". Series 35, Issue 2, Institute of Engineering Research, University of California, Berkeley, California, October 1950.

IV F-4: Morison, J. R. "Moment Distribution on Stepped Caisson". Series 35, Issue 1, Institute of Engineering Research, University of California, June 1950.

IV F-5: Morison, J. R., O'Brien, M. P., and J. W. Johnson; "The Design of Piling Exposed to Wave Action". (unpublished)

IV F-6: Morison, J. R., M. P. O'Brien, J. W. Johnson, and S. A. Schaaf. "The Force Exerted by Surface Waves on Piles". Petroleum Transactions. AIME, Vol. 189; 1950.

IV F-7: Snodgrass, F. E., Rice, E. K., and Hall, M.; "Wave Forces on Piling (Monterey Field Test)", Series 35, Issue 4, Institute of Engineering Research, University of California, Berkeley, California, June 1951.

TABLE IV F-1

SUMMARY OF TEST CONDITIONS
FOR MOMENT DISTRIBUTION ON PILING

| Pile No. | D inches | d feet | Wave Characteristics T sec. | H feet | L feet | H/L | d/L | Remarks |
|---|---|---|---|---|---|---|---|---|
| 1 | 0.5 | 1.96 | 0.98 | 0.184 | 4.91 | 0.0375 | 0.400 | Moments measured at 7 elevations |
| 2 | 1 | 1.96 | 0.98 | 0.179 | 4.98 | 0.0360 | 0.393 | Moments measured at 6 elevations |
| 3 | 2 | 1.96 | 0.98 | 0.186 | 4.96 | 0.0376 | 0.395 | Moments measured at 6 elevations |
| 4 | Variable | 2.24 | 0.86 | 0.406 | 4.18 | 0.0971 | 0.536 | Moments measured about 10 hinge points |

TABLE IV F-2

EFFECT OF PILE SHAPE ON MAXIMUM MOMENT

| Pile type and size Orientation | Ratio: Maximum moment for given pile type / Maximum moment on 1 inch round | | |
|---|---|---|---|
| | H/L = 0.0974 | H/L = 0.0916 | H/L = 0.0741 (Breaker) |
| 1 inch round → O | 1.00 | 1.00 | 1.00 |
| 1 inch H-section → I | 2.02 | 1.62 | 2.36 |
| 1 inch H-section → ⊢ | 2.54 | 1.33 | 2.00 |
| 1 inch H-section → Y 45° | 3.34 | 2.35 | 2.07 |
| 1 inch flat plate → \| | 1.15 | 1.24 | 1.33 |

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TABLE IV F-3

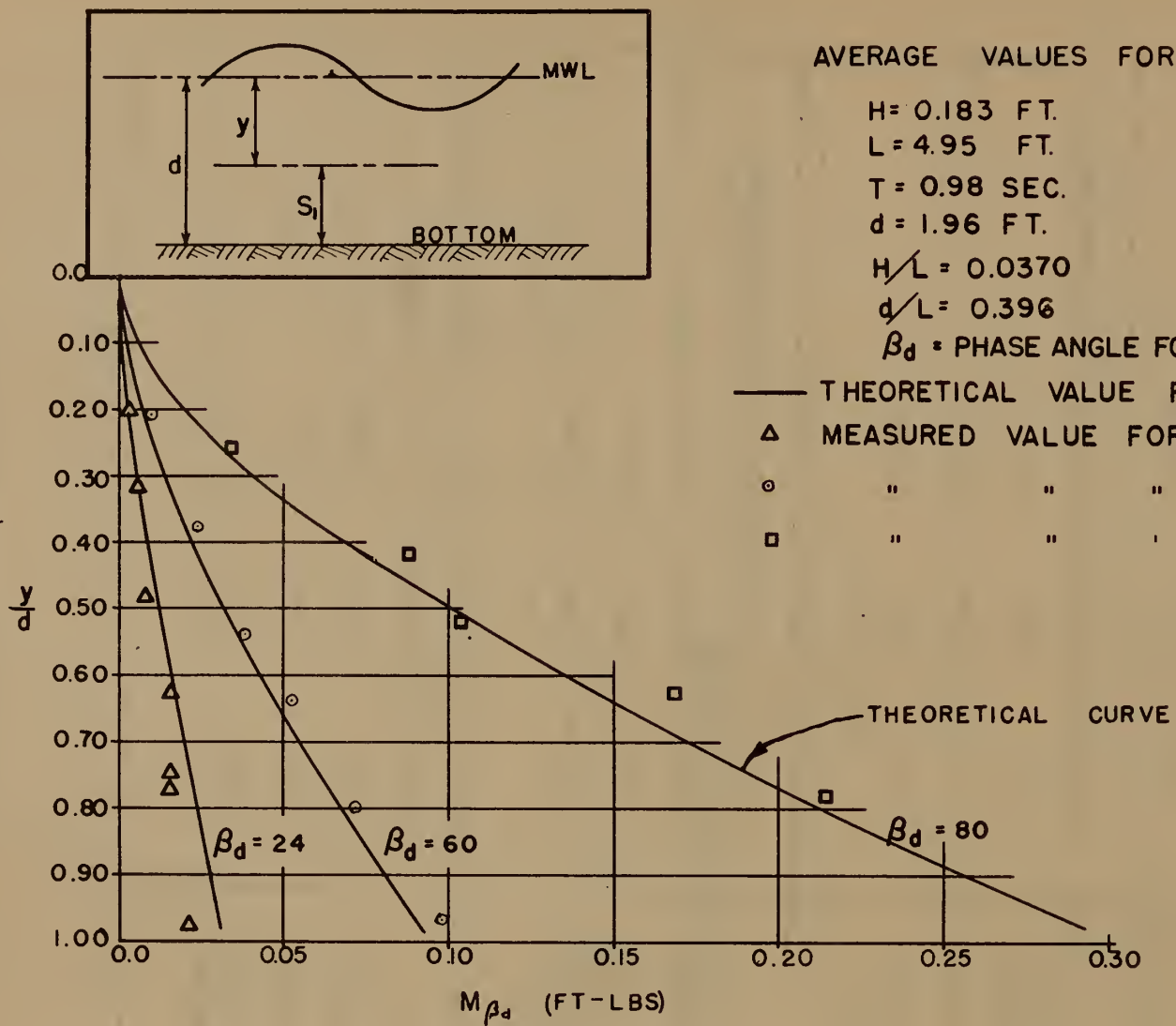
EFFECT OF MUTUAL INTERFERENCE OF PILING

| Wave Steepness | Ratio: $\frac{\text{Moment on center pile}}{\text{Moment on single pile}}$ | | |
|---|---|---|---|
| | Pile gap (a) | | |
| | 1/2 D | D | 1-1/2 D |
| | Round Pile (perpendicular to wave direction) | | |
| H/L = 0.0974 | 2.35 | 0.88 | 0.92 |
| H/L = 0.0916 | - | 1.37 | 1.00 |
| H/L = 0.0741 (Breaker) | 1.44 | 1.03 | 1.06 |
| | Flat plate (perpendicular to wave direction) | | |
| H/L = 0.0974 | 1.64 | 1.19 | 1.00 |
| H/L = 0.0916 | 1.16 | 1.13 | 1.20 |
| H/L = 0.0741 (Breaker) | 1.29 | 1.21 | 0.96 |
| | Round pile (parallel to wave direction) | | |
| H/L = 0.0974 | 0.58 | 0.70 | 0.73 |
| H/L = 0.0916 | 0.35 | 0.68 | 0.73 |
| H/L = 0.0741 (Breaker) | 0.80 | 0.75 | 0.85 |

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AVERAGE VALUES FOR 3 PILES

H = 0.183 FT.
L = 4.95 FT.
T = 0.98 SEC.
d = 1.96 FT.
H/L = 0.0370
d/L = 0.396
$\beta_d$ = PHASE ANGLE FOR TOTAL MOMENT AT DEPTH d.

—— THEORETICAL VALUE FOR VARIOUS PILES
△ MEASURED VALUE FOR $\frac{1}{2}$" PILE. $C_M$=1.15, $C_d$=2.69
⊙ " " " 1" " " =1.83, " =2.54
□ " " " 2" " " =1.79, " =4.41

FIG. IVF-1-- MOMENT DISTRIBUTION ON UNIFORM PILE

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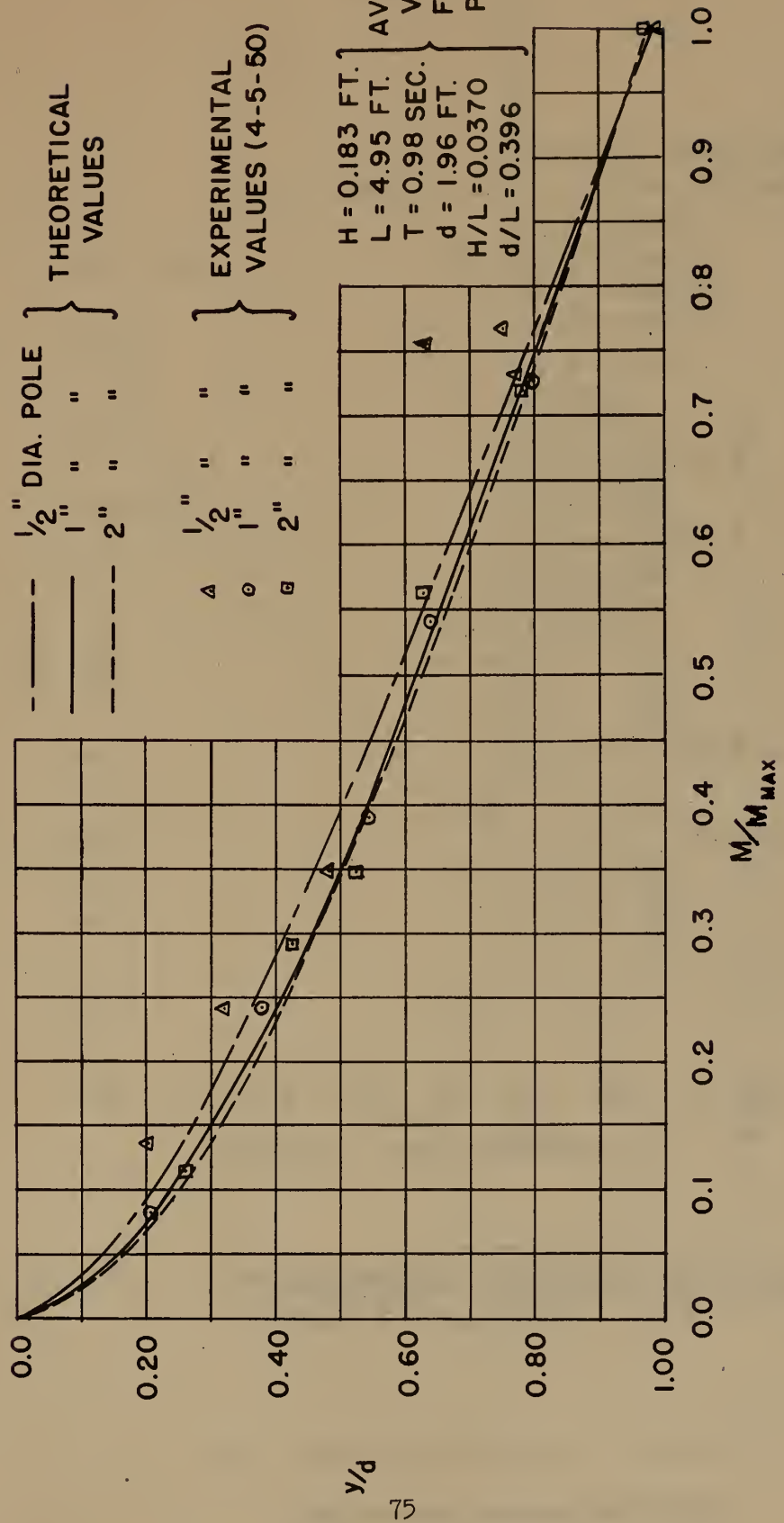



FIG. IV F-2-- DIMENSIONLESS MOMENT DISTRIBUTION OF UNIFORM PILE

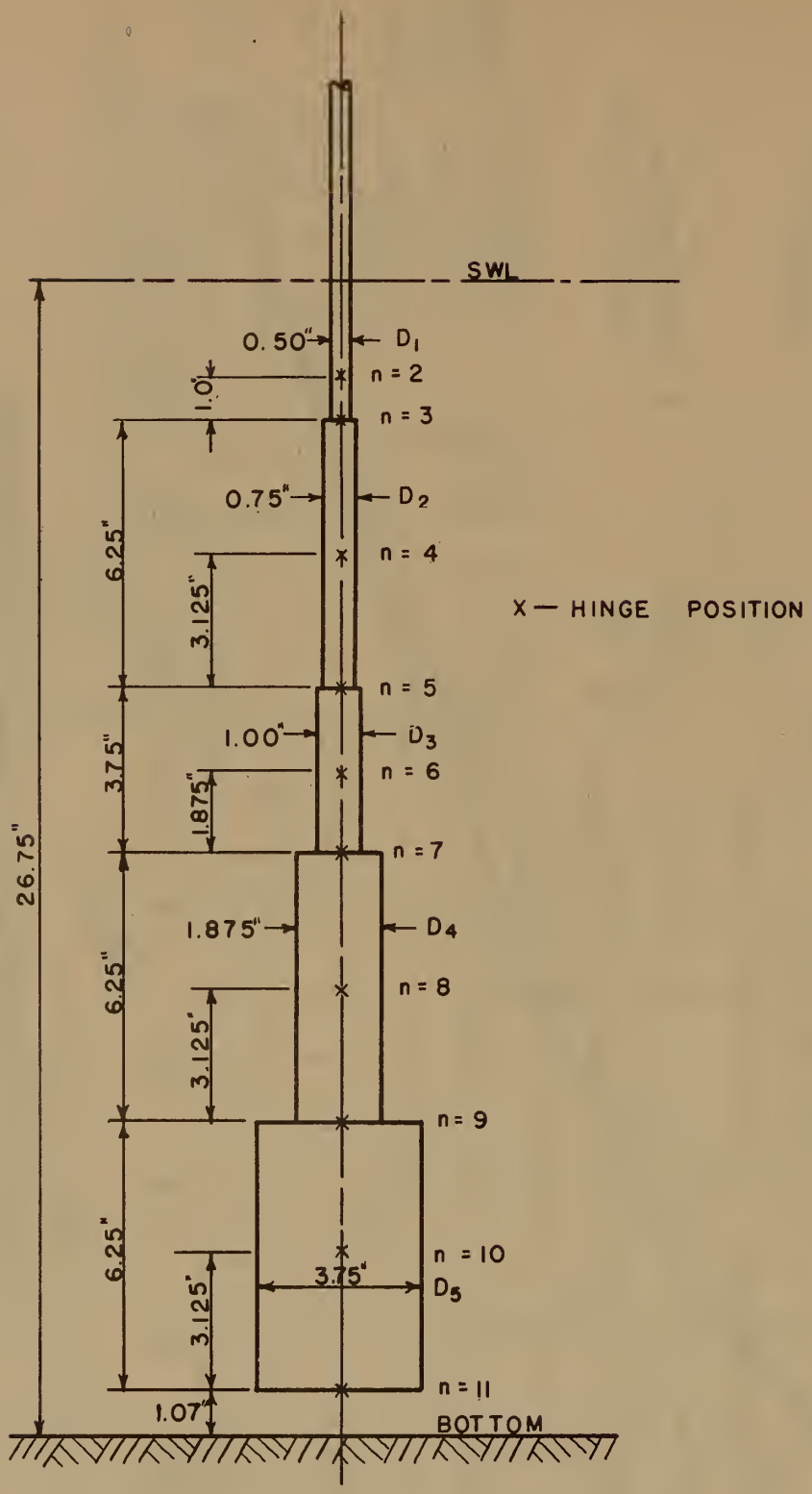



FIG. IVF-3 -- DETAILS OF NON-UNIFORM PILE

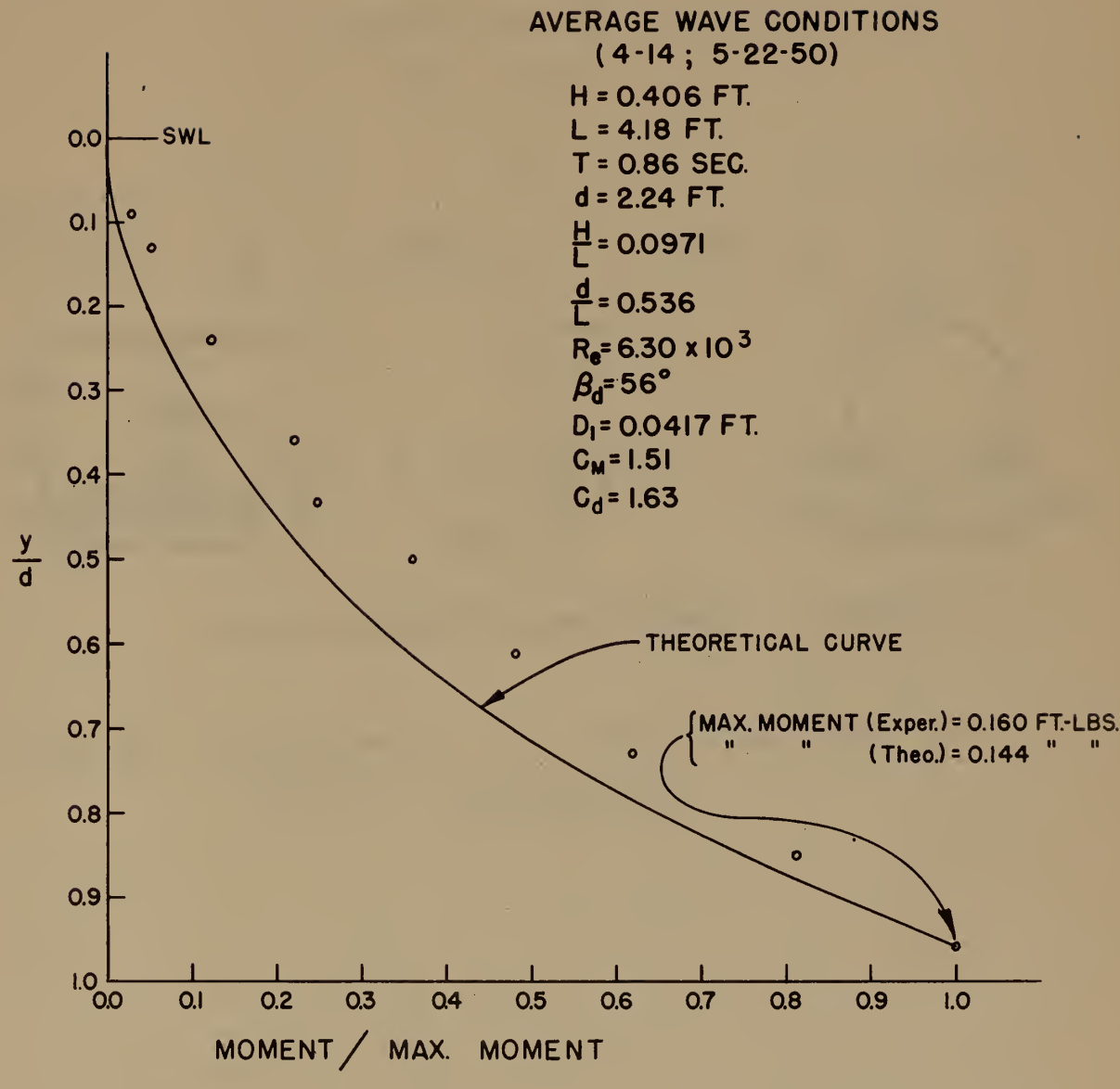



FIG. IV F - 4 -- DIMENSIONLESS MOMENT DISTRIBUTION ON VARIABLE DIAMETER PILE

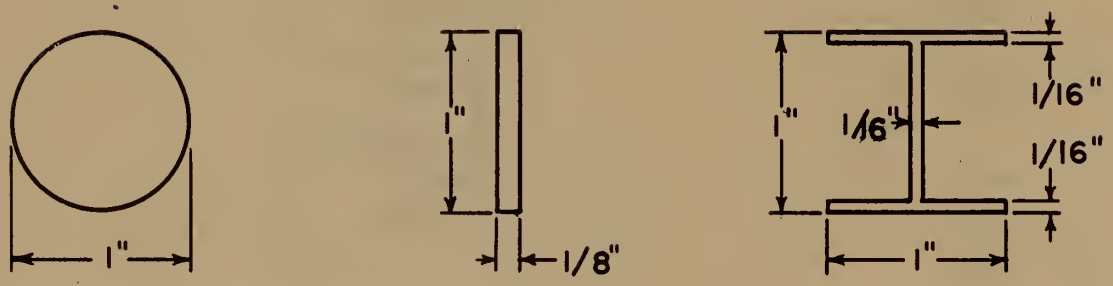
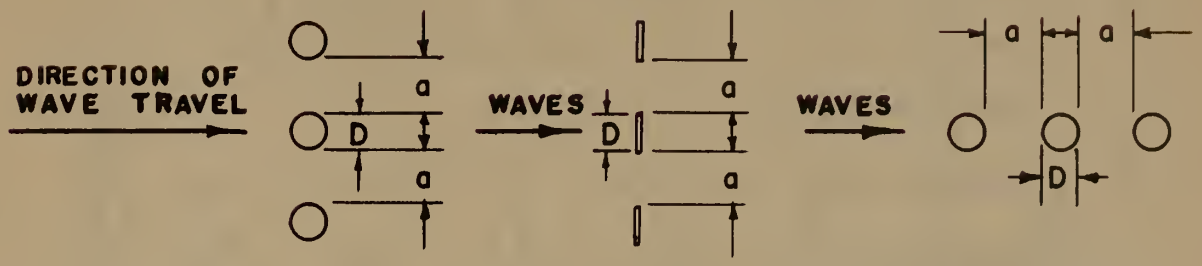



FIG. IV F-5 -- DIMENSIONS OF MODEL PILES

(a VARIED FROM $\frac{1}{2}$" TO $1\frac{1}{2}$")
(D = 1 INCH)




a. ROUND PILING (PERPENDICULAR TO WAVE TRAVEL)

b. FLAT PLATES

c. ROUND PILING (PARALLEL TO WAVE TRAVEL)

FIG. IVF-6 -- ARRANGEMENT OF PILING FOR TEST ON MUTUAL INTERFERENCE

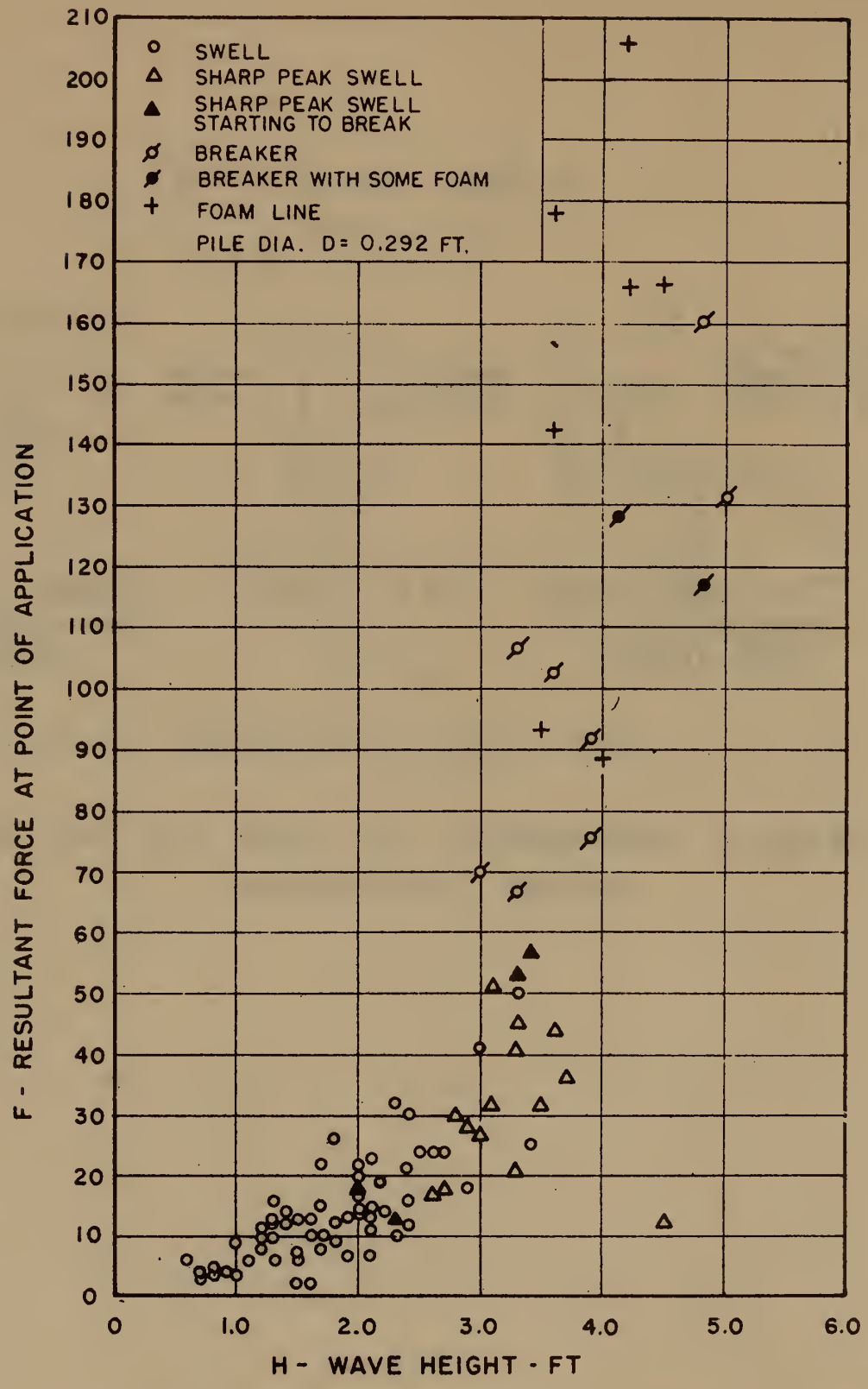



FIG. IVF-7 RESULTANT WAVE FORCE VERSUS WAVE HEIGHT

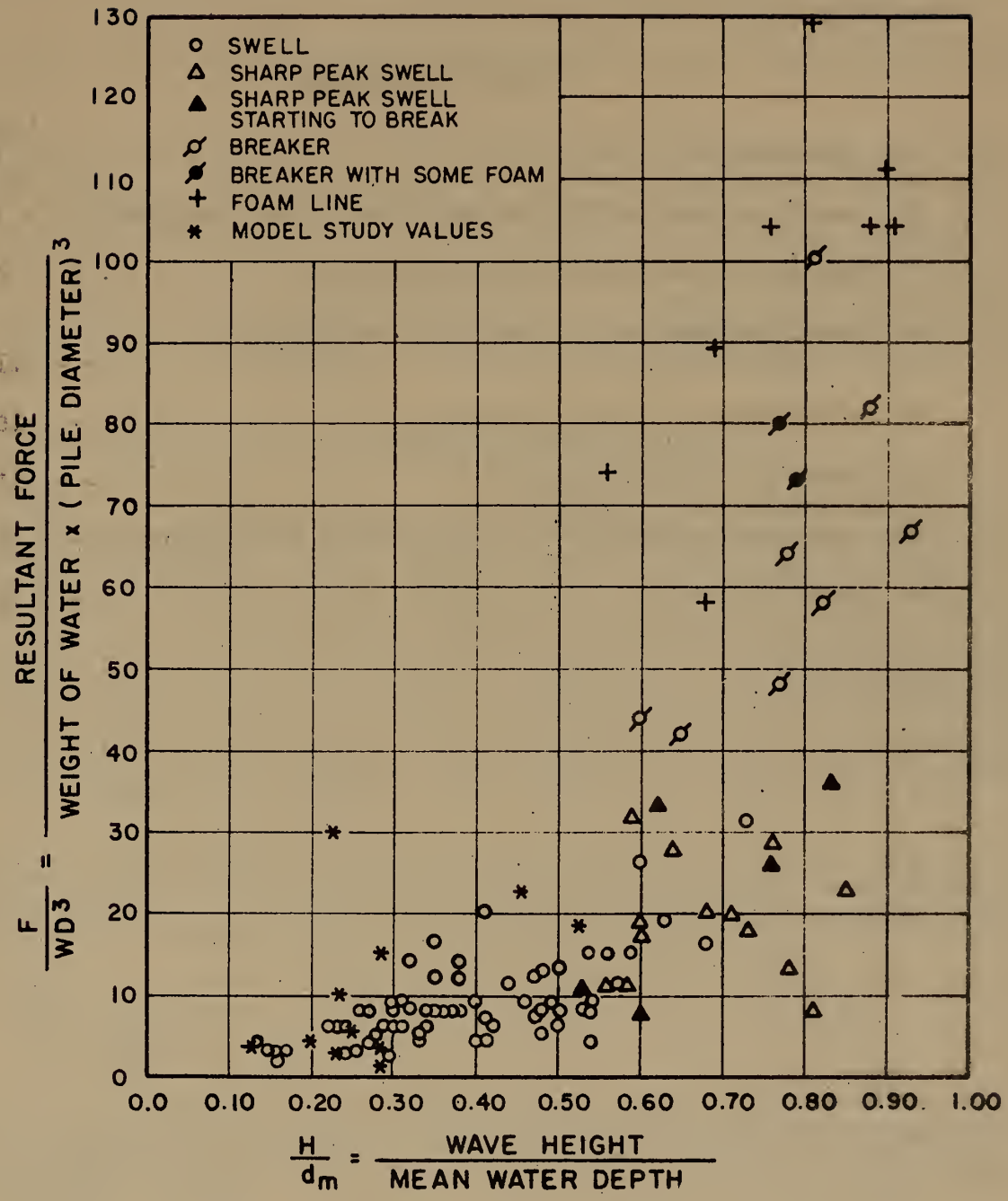



FIG. IV F-8 DIMENSIONLESS FORCE ON PILE VERSUS RATIO OF WAVE HEIGHT TO MEAN WATER DEPTH.

MANUAL OF AMPHIBIOUS OCEANOGRAPHY

SECTION IV: WAVE FORCES ON OBJECTS,
PILING, VERTICAL WALLS AND RUBBLE MOUNDS

# G: WAVE FORCES ON WALLS, FLOATING BREAKWATERS, AND RUBBLE MOUNDS

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# G. WAVE FORCES ON WALLS, FLOATING BREAKWATERS, AND RUBBLE MOUNDS

## 1. Introduction:

The forces exerted by ocean waves on breakwaters are reviewed from the available literature. The subjects covered are the forces caused by incident waves which are reflected as unbroken waves from vertical walls, the forces caused by waves breaking on vertical walls, and the forces exerted on a floating breakwater by waves which are reflected as unbroken waves. Additional subjects covered are the size and weight of the rock in a rubble mound necessary to resist wave action and the slope of a rubble mound that causes the reflection of waves or the breaking of waves on the mound.

Many of the force equations derived from mathematical theories are based on standing waves or reflected waves where the wave is unbroken. The forces resulting from this type of unbroken wave action are not as great as the force exerted by a wave breaking on the structure. The forces of breaking waves can be calculated from empirical equations and some theoretical equations which are difficult to apply and are somewhat complicated. In general, the calculations of wave forces on walls and rubble mounds have been reduced to approximate methods and empirical design data because of the complicated nature of the forces exerted by waves that break on a structure.

## 2. The Forces Exerted by Unbroken Waves on Vertical Walls:

The force of an unbroken wave on a vertical wall is a maximum if the wave is fully reflected. That is, if there is a standing wave sometimes referred to as clapotis wave action. Since almost all walls in the ocean are placed in shallow water, the equations for the pressure on a vertical wall that Sainflou (Reference IV G-12) derived should be used in calculating the pressure of fully reflected waves. This method has been verified for model and prototype structures. The expressions and method of application of the absolute and differential wave pressures were derived by Sainflou from the consideration of the wave theories of Barre de Saint-Venant and Flament and M. Boussinesq. The method is presented in the form in which it is most generally used. This is an approximate form rather than the exact theoretical equation.

Suppose a situation exists where a wave of height H= 6 ft. is fully reflected from a vertical wall. The still water depth, d, is 20 ft. at the wall and the wave length, L, is 100 ft. in the region of the wall. The first step is to calculate the maximum elevation of the mean water level, $h_o$.

$$h_o = \frac{\pi H^2}{L \operatorname{Tanh} \frac{2\pi d}{L}} \qquad \text{(IV G-2.1)}$$

From "Oscillatory Waves", Bulletin of Beach Erosion Board, Special Issue No. 1, July 1, 1948 (Reference IV G-13) or Section II B of this Manual for d/L = 0.200 we find $\operatorname{Tanh} \frac{2\pi d}{L} = 0.8501$

and thus

$$h_o = \frac{\pi 6^2}{100} \frac{1}{0.8501} = 1.33 \text{ feet}$$

Figure IV G-1 shows the graphical construction of Sainflou's approximate method for this example. The maximum elevation of the wave crest at the wall is obtained by adding $d + H + h_o = 27.33$ ft. or $H + h_o = 7.33$ ft. above the still-water level and is plotted as point (1) on Figure IV G-1. The next step is to draw

in the static pressure line, shown as the dotted line from the still-water level (point 2) to the maximum static pressure, d = 20 ft. of water, plotted as point 3. The third step is to compute the maximum amplitude of the dynamic pressure (a) at the bottom of the wall given by the expression

$$a = \frac{H}{\text{Cosh}\frac{2\pi d}{L}} \text{ feet} \qquad \text{(IV G-2.2)}$$

$$= \frac{6}{1.899} = 3.16 \text{ feet}$$

The dynamic pressure is added to the static pressure to give the absolute pressure at the bottom equal to 23.16 feet of water plotted as point 4. Now draw a straight line from point 1 to point 4 to determine the maximum amplitude of the dynamic pressure at the still-water level (point 5). This dynamic pressure can be computed from the trigonometrical relationship

$$p = \frac{(H + h_o)}{(d + H + h_o)}(d+a) \text{ feet} \qquad \text{(IV G-2.3)}$$

$$= \frac{7.33}{27.33}(23.16) = 6.21 \text{ feet of water}$$
$$= 400 \text{ Lbs/Ft}^2$$

The absolute pressure under the crest of the wave on the wall is represented by the line from point (1) to (4). The next step is to plot the amplitude of the dynamic pressure on the bottom (a = 3.16 feet) as point (6) and connect by a straight line points (5) and (6). The curve from points (1) to (5) to (6) represents the maximum dynamic pressure under the crest of the wave on the wall when the wave is unbroken and fully reflected.

The dynamic pressure under the trough of the wave at the wall is constructed to the left on Figure IV G-1 by first drawing in the static pressure, d = 20 ft. from point 2H = 12 ft. below point (1) or 4.67 ft. below the still water level (point 2). Now plot the amplitude of the dynamic pressure (a = 3.16 ft) at the bottom of the wall at point (9). The curve from point (2) to (8) to (9) represents the dynamic pressure under the trough of the wave on the wall. The dynamic pressure under the crest and under the trough as computed above are slightly larger than they would have been if they were calculated from the exact equations. A straight line from point 10 to point 11 represents the absolute pressure on the wall under the trough.

The total dynamic thrust on the wall exerted under the crest of the wave and the resulting total moment about the bottom of the wall are given by the equations

$$R_e = \frac{(d + h_o + H)(d + a)}{2} - \frac{d^2}{2} \qquad \text{(IV G-2.4)}$$

$$= \frac{27.33\ (23.16)}{2} - \frac{20^2}{2} = 316 - 200$$

$$= 116 \text{ feet of water per linearfoot}$$

$$= 7{,}470 \text{ lbs. per linear ft.}$$

and

$$M_e = \frac{(d + h_o + H)^2 (d + a)}{6} - \frac{d^3}{6} \qquad \text{(IV G-2.5)}$$

$$= \frac{(27.33)^2 (23.16)}{6} - \frac{20^3}{6} = 2880 - 1340$$

= 1540 ft. - ft. of water per linear ft.
= 99,200 Ft-lbs. per linear ft.

The point of application of the total thrust

$$\ell_e = \frac{M_e}{R_e} \qquad \text{(IV G-2.6)}$$

$$= \frac{1540}{116} = 13.3 \text{ ft. above the bottom}$$

The total dynamic thrust (or reduction in static head) under the trough and the resulting total moment about the bottom are given by the equation

$$R_i = \frac{d^2}{2} - \frac{(d + h_o - H)(d - a)}{2} \qquad \text{(IV G-2.7)}$$

$$= \frac{20^2}{2} - \frac{(15.33)(16.84)}{2} = 200 - 129$$

= 71 feet of sea water per linear foot
= 4,570 lbs. per linear foot

and

$$M_i = \frac{d^3}{6} - \frac{(d + h_o - H)^2 (d - a)}{6} \qquad \text{(IV G-2.8)}$$

$$= \frac{20^3}{6} - \frac{(15.33)^2 (16.84)}{6} = 1340 - 658$$

= 682 feet of water per linear foot
= 43,800 Ft-Lbs. per linear foot

The point of application

$$\ell_i = \frac{M_i}{R_i} \qquad \text{(IV G-2.9)}$$

$$= \frac{682}{71} = 9.6 \text{ feet above the bottom.}$$

Equations IV G-2.4 to 2.9 were obtained from the trigonometrical relationship of Figure IV G-1. A schematic diagram of the pressures and thrusts is shown in Figure IV G-2.

An example of experimental results compared to the pressure obtained by

Sainflou's method is shown in Figure IV G-3, (Reference IV G-51). The experiment was a model study of the type of caisson breakwater used in the Normandy Invasion. The results were scaled up to full size.

Most authors and authorities have recognized the method suggested by Sainflou for the pressure on a vertical wall exerted by a standing wave in a limited depth water. Other methods for similar applications have been suggested by Benezit, Jorge Lira, and R. R. Minikin but these were not improvements nor were they as exact a solution (References IV G-9 and 11).

Coen Cagli performed experiments to measure the forces of waves on walls and obtained data pertaining to the wave force that might be expected. Experimental results of both Cagli and the author, where applicable, confirmed the results of Sainflou.

A more rigorous mathematical solution for the pressures exerted by standing waves on a vertical wall has been presented by M. Miche (Reference IV G-8). The pressure to the 2nd order, which satisfys all the conditions of an ideal flow with the additional assumption of zero mass transport, is given by the equation:

$$\frac{P}{w g} = y_o + \frac{H \operatorname{Sinh}\frac{2\pi y_o}{L} \operatorname{Sin}\frac{2\pi x_o}{L} \operatorname{Sin}\frac{2\pi t}{L}}{\operatorname{Sinh}\frac{2\pi d}{L} \operatorname{Cosh}\frac{2\pi d}{L}} + \frac{H^2\pi}{4L} \frac{\operatorname{Sinh}\frac{2\pi y_o}{L}}{\left(\operatorname{Sinh}\frac{2\pi d}{L}\right)^2}$$

$$\cdot\left\{\operatorname{Cosh}\frac{2\pi}{L}(2d-y_o) \frac{\operatorname{Cos}\frac{4\pi x_o}{L}}{\left(\operatorname{Cosh}\frac{2\pi d}{L}\right)^2} + 4\operatorname{Sin}^2\frac{2\pi t}{T} + 4\operatorname{Tanh}\frac{2\pi d}{L}\operatorname{Sinh}\frac{2\pi}{L}(2d-y_o)\right.$$

$$\cdot\left[1 - 3\operatorname{Sin}^2\frac{2\pi t}{T}\right] + 3\operatorname{Cos}\frac{4\pi x_o}{L}\operatorname{Cos}\frac{4\pi t}{T}\left[\frac{\operatorname{Cosh}\frac{4\pi y_o}{L}}{\left(\operatorname{Sinh}\frac{2\pi d}{L}\right)^2}\right.$$

$$\left.\left. - \frac{2\operatorname{Cosh}\frac{2\pi}{L}(d-y_o)}{\operatorname{Cosh}\frac{2\pi d}{L}}\right]\right\} \qquad \text{(IV G-2.10)}$$

with the coordinates

$$x = x_o + \frac{H \operatorname{Cosh}\frac{2\pi(d-y_o)}{L} \operatorname{Cos}\frac{2\pi x_o}{L} \operatorname{Sin}\frac{2\pi t}{T}}{\operatorname{Sinh}\frac{2\pi d}{L}}$$

$$- \frac{H^2\pi}{2L} \frac{\operatorname{Sin}\frac{4\pi x_o}{L}}{\left(\operatorname{Sinh}\frac{2\pi d}{L}\right)^2}\left[\operatorname{Sin}^2\frac{2\pi t}{T} + \frac{\operatorname{Cosh}\frac{4\pi}{L}(d-y_o)}{4\left(\operatorname{Sinh}\frac{2\pi d}{L}\right)^2}\left(3\operatorname{Cos}\frac{4\pi t}{T} + \operatorname{Tanh}^2\frac{2\pi d}{L}\right)\right] \qquad \text{(IV G-2.11)}$$

$$y = y_o - \frac{H \operatorname{Sinh}\frac{2\pi(d-y_o)}{L} \operatorname{Sin}\frac{2\pi x_o}{L} \operatorname{Sin}\frac{2\pi t}{T}}{\operatorname{Sinh}\frac{2\pi d}{L}}$$

$$- \frac{H^2\pi}{2L} \operatorname{Sinh}\frac{4\pi(d-y_o)}{L} \left[ \operatorname{Sin}^2 \frac{2\pi t}{T} \frac{\operatorname{Cos} 4\pi x_o}{4(\operatorname{Sinh}\frac{2\pi d}{L})^2} \left( 3\operatorname{Cos}\frac{4\pi t}{T} + \operatorname{Tanh}^2 \frac{2\pi d}{L} \right) \right]$$

(IV G-2.12)

The corresponding theoretical solution obtained by Sainflou is given by the expression:

$$\frac{p}{\rho_w g} = y_o + \frac{H \operatorname{Sinh}\frac{2\pi y_o}{L} \operatorname{Sin}\frac{2\pi x_o}{L} \operatorname{Sin}\frac{2\pi t}{T}}{\operatorname{Sinh}\frac{2\pi d}{L} \operatorname{Cosh}\frac{2\pi d}{L}} \qquad \text{(IV G-2.13}$$

with the coordinates

$$x = x_o + \frac{H \operatorname{Cosh}\frac{2\pi(d-y_o)}{L}}{\operatorname{Sinh}\frac{2\pi d}{L}} \operatorname{Sin}\frac{2\pi t}{T} \operatorname{Cos}\frac{2\pi x_o}{L} \qquad \text{(IV G-2.14)}$$

$$y = y_o - \frac{H \operatorname{Sinh}\frac{2\pi(d-y_o)}{L}}{\operatorname{Sinh}\frac{2\pi d}{L}} \operatorname{Sin}\frac{2\pi t}{T} \operatorname{Sin}\frac{2\pi x_o}{L} - \frac{H^2}{2L} \frac{\operatorname{Sinh}\frac{4\pi(d-y_o)}{L}}{(\operatorname{Sinh}\frac{2\pi d}{L})^2} (\operatorname{Sin}\frac{2\pi t}{T})^2$$

(IV G-2.15)

where x and y are the particle position at which p the pressure occurs and $x_o$ and $y_o$ are the mean position of the particle. The difference between equation IV G-2.10 and 2.13 is the third term of equation IV G-2.10, with the corresponding difference in the coordinates. The meaning of the symbols of equation IV G-2.10 and 2.13 are shown schematically in Figure IV G-4. An example of the pressure computed from Equation IV G-2.10 and 2.13 as compared to experimental results by M. Stucky is shown in Figure IV G-5 (Reference IV G-8). From Figure IV G-5, the difference between equation IV G-2.10 and 2.13 is shown to be only a slight amount. The equation IV G-2.10 by Miche seems to agree to the experimental result more closely than Sainflou's method, thus making the approximate method of Sainflou somewhat higher still than the experimental results. It is sufficient to say that Miche's equation was obtained by satisfying the equation of motion and the equation of continuity and satisfying the conditions of zero pressure at the surface, zero vertical velocity at the bottom and zero mass transport. The results can be expressed by a potential function or stream function using either Lagrange or Euler variables as the results also satisfy the conditions for irrotational flow.

Simple verification of Sainflou's equation can be derived from modern hydrodynamics. Considering the Euler variable of water particles velocity, the second order potential and stream functions are:

$$\phi = \frac{HL}{T}\frac{\cosh\frac{2\pi(d-y)}{L}}{\sinh\frac{2\pi d}{L}}\sin\frac{2\pi x}{L}\cos\frac{2\pi t}{T} - \frac{3\pi H^2}{8T}\frac{\cosh\frac{4\pi(d-y)}{L}}{\left(\sinh\frac{2\pi d}{L}\right)^4}\cos\frac{4\pi x}{L}\sin\frac{4\pi t}{T} \qquad \text{(IV G-2.16)}$$

$$\psi = \frac{-HL}{T}\frac{\sinh\frac{2\pi(d-y)}{L}}{\sinh\frac{2\pi d}{L}}\cos\frac{2\pi x}{L}\cos\frac{2\pi t}{T} - \frac{3\pi H^2}{8T}\frac{\sinh\frac{4\pi(d-y)}{L}}{\left(\sinh\frac{2\pi d}{L}\right)^4}\sin\frac{4\pi x}{L}\sin\frac{4\pi x}{T} \qquad \text{(IV G-2.17)}$$

We now consider the Bernoulli equation for the potential flow given by the expression

$$\frac{p}{\rho_w g} = \frac{1}{g}\frac{\partial\phi}{\partial t} = y \qquad \text{(IV G-2.18)}$$

where y can be obtained by successive approximations from equation IV G-2.17. What Sainflou actually did was to consider the equations of the particle position (x, y) for standing waves in limited depth of water to the second order. He substituted these equations into the equation of continuity and the equation of equilibrium and obtained the expressions given by equations IV G-2.14 and 2.15.

By substituting equation IV G-2.14 and 2.15 into equation IV G-2.16 and substituting the result along with equation IV G-2.15 into equation IV G-2.17 and retaining only first order terms, we obtain the expression presented by Sainflou

$$\frac{p}{\rho_w g} = y_o \pm H\sin\frac{2\pi t}{T}\sin\frac{2\pi x_o}{L}\left(\frac{\cosh\frac{2\pi(d-y_o)}{L}}{\cosh\frac{2\pi d}{L}} - \frac{\sinh\frac{2\pi(d-y_o)}{L}}{\sinh\frac{2\pi d}{L}}\right) \qquad \text{(IV G-2.19)}$$

If equation IV G-2.19 is simplified it becomes equation IV G-2.13. The pressure, p, is acting at the position x, y. Along a wall, the maximum pressure occurs when $\sin 2\pi x_o/L = \pm 1$ and has its line of action at the depth y. If we desire the differential pressure caused by the wave we substract from p the hydrostatic pressure due to the still water. This may easily be done; however, it is better to draw a graph.

3. Force of Waves that Break on Walls:

Another approach to the solution of wave force on walls including breaking waves was made by Col. D. D. Gaillard (Reference IV G-31) and somewhat extended by David A. Molitor (Reference IV G-11). This method was based on the pressure caused by a maximum water particle velocity and wave velocity adjusted by an empirically determined coefficient. The pressure is given by the expression:

$$p = \frac{f\, w\, A\, (C + u)^2}{2g} \qquad \text{(IV G-3.1)}$$

where

f = constant - 1.25 to 2.0 (1.31 for Great Lakes)
w = water 64.4 #/ft$^3$
A = area ft$^2$
C = wave velocity ft/sec
u = orbital velocity ft/sec
g = 32.2 ft/sec$^2$

Considering only shallow water, Gaillard used the expression:

$$C = \sqrt{\frac{5.12\, b_s L}{a_s}}$$

$$u = a_s \sqrt{\frac{b_s\, 2\pi g}{a_s\, L}}$$

where

$$a_s = b_s \frac{1}{\text{Tanh } \frac{2\pi d}{L}}$$ = Semi major axis of surface particle - feet.

$b_s = \frac{H}{2}$ = Semi-minor axis of surface particle - feet.

He found the maximum pressure to be at the still water level. From experiments with prototype structures, he obtained pressure as high as 4,000 to 6,000 pounds per square foot.

Molitor used the same equations as Gaillard except for the expressions

$$a_s = b_s \frac{\exp\left(\frac{4\pi d_o}{L}\right) + 1}{\exp\left(\frac{4\pi d}{L}\right) - 1}$$

$$\frac{d_o}{L} = \frac{d}{L} + 2\left(\frac{H}{L}\right)^2$$

He used the empirical constant f equal to 1.8 for ocean waves.

He found that the maximum pressure was a distance $h_1 = 0.12H$ above the still-water level and concluded that no dynamic pressure occurs below the trough. This latter statement about the trough has been shown by experiment to be incorrect.

Suppose, as previously, that the wave height is 6 ft. and the wave length is 100 ft. for a vertical wall where the still-water depth is 20 ft. Assume that $f = 1.31$. We now compute the pressure exerted on one square foot of wall at the still-water level by Gaillard's method.

$$b_s = \frac{H}{2} = 3 \text{ ft.}$$

$$a_s = b_s \frac{1}{\text{Tanh } \frac{2\pi d}{L}} = \frac{3}{0.8501} = 3.53 \text{ ft.}$$

$$C = \sqrt{\frac{5.12\, b_s L}{a_s}} = \sqrt{\frac{5.12 \times 3 \times 100}{3.53}} = 20.9 \text{ ft/sec}$$

$$u = a_s \sqrt{\frac{b_s\, 2\pi g}{a_s\, L}} = 3.53 \sqrt{\frac{3 \times 2 \times \pi \times 32.2}{3.53 \times 100}} = 4.63 \text{ ft/sec}$$

and from equation IV G-3.1

$$p = \frac{f\, w\, A\, (C + u)^2}{2g} = \frac{1.31 \times 64.4 \times 1 (20.9 + 4.63)^2}{2 \times 32.2}$$

$$= 1.31\ (25.53)^2 = 851 \text{ lbs/ft}^2 \text{ where A = 1 square foot.}$$

This pressure is considerably higher than that found by Sainflou's method. For the same problem, the method of Molitor gives the following results.

$$\frac{d_o}{L} = \frac{d}{L} + 2\left(\frac{H}{L}\right)^2 = 0.200 + 2\left(\frac{6}{100}\right)^2 = 0.2072$$

$$a_s = b_s \frac{\exp (4\pi d_o/L) + 1}{\exp (4\pi d_o/L) - 1} = 3 \frac{\exp (4\pi\ 0.207) + 1}{\exp (4\pi\ 0.207) - 1} = 3 \frac{13.4 + 1}{13.4 - 1} = 3\ \frac{14.4}{12.4}$$

$$= 3.5 \text{ ft.}$$

$$C = \sqrt{\frac{5.12 \times 3 \times 100}{3.5}} = 21.0 \text{ ft/sec}$$

$$u = 3.57 \sqrt{\frac{3 \times 2 \times \pi \times 32.2}{3.5 \times 100}} = 4.7 \text{ ft/sec}$$

and from equation IV G-3.1

$$p = \frac{1.31 \times 64.4 \times 1\ (21.0 - 4.7)^2}{2 \times 32.2} = 1.31 \times (25.7)^2$$

$$= 865 \text{ lbs/ft}^2 \quad \text{where A = 1 square foot}$$

This pressure is only slightly different than that obtained by Gaillard's method but is about twice that obtained by Sainflou's method.

An example of measured results at the site and a model of Prince Humbert Jetty, Geneva compared to Sainflou's method and Molitor's method is shown in Figure IV G-6 (Reference IV G-11). Sainflou's method agrees very well with the measured results at the site and to the model results of Coen Cagli. Molitor's method gave a high result because it takes into account the impact of a breaking wave and when a wave does not break on a wall this method does not apply.

Major B. A. Bagnold (Reference IV G-1) by experiment and theoretical analysis has examined the problem of the impact pressure of waves that break on a vertical wall. He considered the pressure obtainable by a pocket of air trapped between the wave and the wall analagous to the compression of a short stroke piston. The momentum balance is expressed by

$$\rho_w \, CK = \int p \, dt \qquad \text{(IV G-3.2)}$$

where

$C$ = velocity of water
$K$ = length of the water slug
$\rho_w$ = density of water - 2.0 slugs/ft$^3$
$p$ = pressure
$K_{max} = \frac{H}{5}$ approximately for breaking wave

Motion of the piston is given by

$$p = \rho_w K \frac{\partial^2 x}{\partial t^2} + p_o \qquad \text{(IV G-3.3)}$$

Pressure of entrapped air is

$$p = p_o \ell^r x^r \qquad \text{(IV G-314)}$$

Assuming the compression adiabetic and by substituting into equation IV G-3.3 integrating, the maximum ualue of the pressure is given within ± 10% by the expression

$$p_{max} - p_o = 0.54 \rho_w \, C^2 \frac{H}{\ell} \qquad \text{(IV G-3.5}$$

where

$\ell$ = thickness of air cushion
$C^2 = \frac{gL}{2\pi} \operatorname{Tanh} \frac{2\pi d}{L}$
$H$ = wave height
$p_o$ = atmospheric pressure

The results of the experiments fixed the maximum pressure at the still water level and of an amplitude of about 10 times that obtained for full reflection.

A comparison of the maximum pressure of a breaking wave against vertical wall to the pressure as calculated by Bagnold's equation IV G-3.5 was obtained by a model study. Figure IV G-7 shows the wave breaking on the wall and the pressure record. From Frame 3, Figure IV G-7, it was possible to measure the

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thickness of the entrapped air, $\ell = 0.020$ ft. In this problem the wave height was 0.210 ft, the wave length was 6.0 ft. and the still-water depth was 0.250 ft. First, the wave velocity is computed for $d/L = 0.042$.

$$C^2 = \frac{gL}{2\pi} \text{Tanh} \frac{2\pi d}{L} = \frac{32.2 \times 6.0}{2\pi} \times 0.2579$$

$$= 7.92 \left[\text{ft/sec}\right]^2$$

From equation IV G-3.5

$$(p_{max} - p_o) = 0.54 \rho_w C^2 \frac{H}{\ell} = 0.54 \times 2 \times 7.92 \frac{0.210}{0.020}$$

$$= 90.0 \text{ lbs/ft}^2$$

$$= 1.4 \text{ ft. of water}$$

From Figure IV G-7, the time history of pressure shows a pressure of about 0.41 ft. of water followed by a pressure of about 0.25 ft. of water. These pressures are considerably less than the computed peak pressure of 1.4 ft. of water. The record shows that a higher maximum pressure was probably experienced but not recorded by the instrument.

By Sainflou's method, the peak pressure of the model wave if fully reflected could be obtained by using equations IV G-2.1, 2.2 and 2.3

$$h_o = \frac{\pi H^2}{L \text{ Tanh } \frac{2\pi d}{L}} = \frac{\pi (0.210)^2}{6 \times (0.2579)} = 0.0896 \text{ feet}$$

$$a = \frac{H}{\text{Cosh} \frac{2\pi d}{L}} = \frac{0.210}{1.035} = 0.203 \text{ feet}$$

$$p = \frac{(H + h_o)}{d + H + h_o} (d + a) = \frac{(0.2996)(0.453)}{0.5496} = 0.2465 \text{ feet}$$

This hydrostatic pressure is about the magnitude of the pressure which followed the impact and is only about 1/6 of the calculated impact pressure.

The time history pressure records of model studies of impact pressure by M. J. Larras, (Reference IV G-6) R. A. Bagnold and J. R. Morison (Reference IV G-10) are shown in Figure IV G-8. These records show that the impact pressure is much greater than the pressure that immediately follows. This latter pressure is approximately equal to the maximum pressure of a standing wave or a fully reflected wave that was unbroken, that is the pressure that would be calculated by Sainflou's method.

If the model results given above are scaled up lineally to the full size problem where the wave height is 6 ft., the wave length is 100 ft. and the water depth is 20 ft., the scale factor would be $6.0/0.21 = 28.6$. This scale factor is the ratio of the prototype wave height to the model wave height. The wave height is considered the most important factor that contributed to the thickness of the air pocket and hence to the peak pressure. Thus, the air pocket thickness becomes 0.572 ft. The wave velocity is

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$$C^2 = \frac{32.2 \times 100}{2\pi} \times 0.8501 = 446.0 \left[\text{ft/sec}\right]^2$$

and from equation IV G-3.5

$$(p_{max} - p_o) = 0.54 \times 2 \times 446.0 \times \frac{6}{0.572} = 5{,}060 \text{ Lbs/Ft}^2$$

This approximate peak pressure is considerably greater than that calculated by Gaillard's method and over 12 times that calculated by Sainflou's method. It should be remembered that this impact pressure acts instantaneously while the hydrostatic pressure may be sustained for about 0.10 to 0.20 or more of the wave period.

R. R. Minikin (Reference IV G-9) considered the work of Bagnold, equation IV G-3.5, and coordinated some of the factors with wave theory. His object was to eliminate the term $\frac{1}{2}$ of equation IV G-3.5 and include the effect of the beach slope in front of the breakwater. This he did by considering the shock momentum of the wave and thus the kinetic energy. He equated this term to the equivalent part of Bagnolds equation and obtained the following expression

$$\rho_w \, CK = I = \rho_w \, \frac{2\pi d}{L} \frac{H}{d_1} \sqrt{g\left(\frac{d_1 + d}{2}\right)} \qquad \text{(IV G-3.6)}$$

where

$$C = \sqrt{g\left(\frac{d_1 + d}{2}\right)} \text{ Ft/Sec - were volocity}$$

$d_1$ = depth of water one wave length from wall - feet.

Now the maximum impulse pressure is given by

$$p_{max} = mIC \qquad \text{(IV G-3.7)}$$

where m = 1 for smooth vertical wall. Hence the maximum impulse pressure is

$$p_{max} = \frac{w\pi g \, H \, d \, (d_1 + d)}{L \, d_1} \text{ Lbs/Ft.}^2 \qquad \text{(IV G-3.8)}$$

and this pressure occurs at the still water level. For the numerical example where H = 6 ft., L = 100 ft., and d = $d_1$ = 20 ft. we have that

$$p_{max} = \frac{62.4\pi \times 32.2 \times 6 \times 20 \times (20 + 20)}{100 \times 20} = 15{,}100 \text{ lbs/ft}^2$$

at the still water level. This result is considerably higher than that obtained by Sainflou's, Gaillard's, or Bagnold's method. The magnitude is over 37 times that given by Sainflou's method and 3 times that given by Bagnold's method.

4. The Force Exerted on Floating Breakwaters by Reflected Waves:

In the Normandy invasion (WWII) a series of floating sections were used as an outer breakwater. This structure was known as the "Bombardon" floating breakwater (See Figure IV G-9) (Reference IV G-7). They were used to protect the inner breakwater while it was under construction and to provide a semi-protected area between the outer and inner breakwaters. The breakwater was designed for waves 8 ft. high and 100 ft. long and later for waves 10 ft. high and 150 feet long.

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The design of a floating breakwater is based upon four principles of wave action and floating bodies. They are 1) that waves for a given location can be accurately forecast; 2) that the majority of the wave action of the sea is relatively close to the surface; 3) the amplitude of a floating body with a long natural period is small when the wave train has a relatively short period; and 4) a floating breakwater carefully designed may have long natural periods in each of its three modes of oscillation; rolling, pitching, and heaving.

The problem of a floating breakwater may be considered in two parts, the design for a long natural period and the wave force from reflected waves. The objective is to minimize the transmitted wave landward of the breakwater; the "transmitted" wave being caused by the motion of the breakwater and the passage of wave energy under the breakwater. The reason for designing a breakwater with a long natural period was so that it would remain relatively stationary and act as a reflector with only slight transmission. A test performed previous to the invasion showed they were satisfactory. In the Normandy invasion, these breakwaters served their purpose well. In the test gale where the waves were 8 ft. high and 150 ft. long the waves within the breakwater were only 2 ft. high. It is important to examine the size of the floating breakwater in relationship to its effectiveness. The above example showed effective results on waves of period less than 5 to 6 seconds. Would this type breakwater be effective for longer wave periods? By use of analogies, experiments, and theory it can be shown that it would not be effective against long period waves. First of all, it is known that wave damping devises such as permeable barriers, air breakwaters, and underwater barriers are most effective against short period waves. By model experiment and theoretical studies, the range of wave periods of waves that are reflected with little transmission is limited for a given size of floating body. As the wave period increases the effectiveness decreases, thus for longer period waves than 6 seconds, a larger breakwater with a longer natural period than the "Bonbardon" float would be necessary.

The theoretical approach to the problem of how to obtain the long natural period for a breakwater is made by considering the equation of motion of a floating body and analyzing the component parts.

The equation of motion of the floating body is

$$m \frac{d^2 s}{dt^2} + Q \frac{ds}{dt} + Rs = a \cos \frac{2\pi t}{T} \qquad \text{(IV G-4.1)}$$

where the sclution

$$s = b \cos\left(\frac{2\pi t}{T} - \beta\right) \qquad \text{(IV G-4.2)}$$

when

$$b = \frac{a}{R\sqrt{\left(1 - \frac{T_N^2}{T^2}\right)^2 + \frac{Q}{mR}\frac{T_N^2}{T^2}}} \qquad \text{(IV G-4.3)}$$

$$\tan \beta = \frac{2\pi Q}{R\,(T^2 - T_N^2)} \qquad \text{(IV G-4.4)}$$

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s = displacement of body about its mean position
T = force period
$T_N$ = natural period of body
m = mass of body
Q = damping coefficient
R = elasticity coefficient
a = forced amplitude
b = amplitude of the body
$\rho$ = phase angle

If $T_N$, the natural period of the body, is long compared to the force period then the amplitude of the body, b, will be small, hence the transmitted wave, generated by the body will be small. Also if the $Q^2/mR$ relationship, or namely Q, the coefficient of damping, is as large as possible for a particular body floating in water then the amplitude b will be small. This requires considerable ingenuity since the mass must be large and also R, the elasticity coefficient, should be as large as possible but, in addition the body must float. Thus, the problem is to build a float with considerable mass, low restoring force, large surface and shape resistance, and a long natural period for the three modes of oscillation; rolling, pitching, and heaving. Another inherent difficulty is that the breakwater must float high enough out of the water so that waves will not top it. One solution to the problem was a structure with flexible sides. This idea was abandoned due to the vulnerability of such a structure in that it had a covering somewhat like a dirigible. The final design was a cross shaped outer structure using water inside to make up the mass. Thus, a large mass was obtained with a low restoring force and displacement proportional to the weight of the encasing structure. The cross-sectional shape of the structure permitted it to float with enough section above water to prevent waves from tipping it. This cross-section also gave high resistance or damping effect. It was a very successfull structure. Further discussion of this structure and its use appears in Section IX "Invasion Harbors", of this Manual.

The second part of the floating breakwater problem is the horizontal thrust of the waves that are reflected by the barrier, and cause a difficult anchorage problem. If the floating breakwater surges, it will generate waves in the protected area. The theoretical solution for the amount of thrust was based on the assumption 1) that the wave amplitude was small and the water is deep, 2) that the incident and reflected wave differed in amplitude, 3) that barrier did not extend to the bottom, 4) that the barrier had a finite mass and was floating with some surging, and 5) that hydrostatic pressure exerted the force on the barrier. Considering the equations of motion of an incident and reflected wave, the equation of motion of the body and the energy of the waves, the theory of Lachner, Faber, and Penny is summarized as follows: (Reference IV G-7)

From the equation of equilibrium

$$\frac{\partial p}{\partial y} + \rho_w g = -\rho_w \ddot{\bar{y}} \qquad \text{(IV G-4.5)}$$

where

p = pressure at the particle position x, y.
$\rho_w$ = water density
g = gravitation constant
$\bar{x}$ = particle displacement in the x-direction
y = particle displacement in the y-direction
$x_o$ = horizontal mean particle position measured from still-water position.

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$y_o$ = vertical mean particle position measured negatively downward from still-water.
$\omega$ = angular velocity of the particle
$t$ = time

Figure IV G-10 shows a schematic diagram of the significance of the symbols. Since $\bar{y}$ is harmonic function $\ddot{\bar{y}} = -\omega^2\bar{y}$; thus, equation IV G-4.5 becomes

$$\frac{\partial p}{\partial y} + \rho_w g = \rho_w \omega^2 \bar{y} \qquad \text{(IV G-4.6)}$$

By integration

$$p + p_o = -\rho_w g y_o + \rho_w \omega^2 \int \bar{y} dy + F \qquad \text{(IV G-4.7)}$$

The integration factor F is independent of $x_o$ and $y_o$ and is identified with $p_o$. Letting p be the excess pressure or dynamic pressure we have

$$p = -\rho_w g y_o + \rho_w \omega^2 \int \bar{y}\, dy \qquad \text{(IV G-4.8)}$$

Now for $\bar{y}$ of the form $\exp(Ky_o)\sin(Kx_o + \omega t)$

$$\int \bar{y}\, dy = \bar{y}/K \qquad \text{(IV G-4.9)}$$

and $\omega^2 = Kg$, and $K = \frac{2\pi}{L}$, and also $\omega = \frac{2\pi}{T}$

hence equation IV G-4.8 becomes

$$p = -\rho_w g y_o + \rho_w g \bar{y} \qquad \text{(IV G-4.10)}$$

To obtain the pressure on the varrier at any depth $y_o$ on the incident wave side we substitute in for $\bar{y}$ the expression for the sum of the incident and reflected waves.

$$\bar{y}_R = e^{Ky_o}\left[a_1 \sin(Kx_o - t) - a_2 \sin(Kx_o - t)\right] \qquad \text{(IV G-4.11)}$$

where

$a_1$ = amplitude of incident wave
$a_2$ = amplitude of reflected wave
$a_3$ = amplitude of transmitted wave

and by energy relationship

$$a_1^2 = a_2^2 + a_3^2 \qquad \text{(IV G-4.12)}$$

If the barrier extends to the depth, $l$, the total pressure is

$$\int_{l}^{\bar{y}_{RD}} p\, dy = \rho_w g\left[\frac{l^2}{2} + \frac{1}{K}(1-e^{-Kl})\left\{a_1\sin(Kx_o+\omega t) - a_2\sin(Kx_o-\omega t)\right\}\right] \qquad \text{(IV G-4.13)}$$

where $\bar{y}_{RD}$ is the evaluation of $\bar{y}_R$ at $y_o = 0$.

Now by a similar process the total pressure on the transmission side of the barrier is

$$\rho_w g\left[\frac{l^2}{2} + \frac{1}{K}(1-e^{-Kl})\, a_3 \sin\left(\omega t - \frac{\pi}{2} - Kx_o\right)\right] \qquad \text{(IV G-4.12)}$$

The resultant force on the barrier is obtained as the difference of equation IV G-4.11 and 4.12, and by the substituting into equation IV G-4.12 of the relationship of the surface profiles of incident, reflected and transmitted wave we obtain the final result for the total dynamic thrust on the floating barrier.

$$p = \frac{2\rho_w g}{K}(1-e^{-Kl})\, a_2 \sin(Kx_o - \omega t) \qquad \text{(IV G-4.13)}$$

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This pressure is the hydrostatic pressure caused by the amplitude of the reflected wave, for the maximum pressure Sin $(Kx_o - \omega t) = 1$. In addition, it can be shown that there if no steady pressure on the floating breakwater for reflection unless some energy of the wave is absorbed by the barrier; that is, it a wave broke on the barrier.

Suppose the floating barrier is subject to a wave that is 8 ft. in height and 150 ft. in length and that the transmitted wave is 2 ft. in height. The barrier extends 19 ft. below the still-water level and the barrier is in 72 ft. of water. From equation IV G-4.13 the maximum total thrust becomes

$$P = -\frac{\rho_w g L}{\pi}\left(1 - e^{-\frac{2\pi\ell}{L}}\right)a_2 \qquad \text{(IV G-4.14)}$$

to obtain $a_2$ we use equation IV G-4.12

$$a_1^2 = a_2^2 + a_3^2$$

$$a_2 = \sqrt{a_1^2 - a_3^3} = \sqrt{4^2 - 1^2} = 3.88 \text{ Ft.}$$

Thus

$$P = -\frac{2.0 \times 32.2 \times 150}{\pi}\left(1 - e^{-\frac{2\pi 19}{150}}\right) 3.88$$

$$= -11{,}900\ (1 - e^{-1.796}) = -11{,}900\ (1 - \tfrac{1}{250})$$

$$= -11{,}900\ (1 - 0.4) = -7{,}150 \text{ Lbs. per linear foot}$$

Now suppose we use Sainflou's method (Reference IV G-2) for a vertical wall but consider only that portion of the wall to the depth of $\ell = 19$ ft. From Figure IV G-11 we see that the total thrust becomes

$$R_e = \frac{(\ell + h_o + H)}{2}(d + a') - \frac{\ell^2}{2} \qquad \text{(IV G-4.15)}$$

where

$$(\ell + a') = \left(\frac{\ell + h_o + H}{d + h_o + H}\right)(d + a) \qquad \text{(IV G-4.16)}$$

Let H = 8 ft.

From equation IV G-2.1 where $d/L = 72/150 = 0.48$

$$h_o = \frac{\pi H^2}{L \tanh \frac{2\pi d}{L}} = \frac{\pi 8^2}{150\ (0.9952)} = 1.35 \text{ ft.}$$

From equation IV G-2.2

$$a = \frac{H}{\cosh \frac{2\pi d}{L}} = \frac{8}{10.23} = 0.782$$

Therefore

$$(\ell + h_o + H) = 19 + 1.35 + 8 = 28.35 \text{ ft.}$$
$$(d + h_o + H) = 72 + 1.35 + 8 = 81.35 \text{ ft.}$$
$$(d^o + a) = 72.78 \text{ ft.}$$

$$\ell + a' = \frac{28.35 \times 72.78}{81.35} = 25.4$$

and

$$R_e = \frac{(28.35)\ (25.4)}{2} - \frac{19^2}{2} = 359 - 180$$

= 179 feet of water per linear foot
= 11,500 lbs. per linear foot

This value of total thrust is considerably greater than that computed by the method derived by Lochner, Faber, and Penney.

5. Discussion of Floating Breakwaters:

The comparison of the total thrust by reflected waves from a floating breakwater to that of a fixed vertical wall indicates that if a floating breakwater is anchored so that it does not surge the total thrust may be more than that calculated by Lochner, Faber and Penney. Furthermore, if a wave should break on the structure the total thrust would be even greater.

The theory was extended and by considering the motion of the barrier, the force distribution, and the moments it was possible to find the line of action of the total thrust which is given by the expression (Reference IV G-7).

$$\ell_a = -\frac{\ell e^{-K\ell}}{1-e^{-K\ell}} + \frac{1}{K} \qquad \text{(IV G-5.1)}$$

In addition it was found that if the wave strikes the barrier at an angle that the total thrust was less and that the transmission was less than for waves striking the barrier where the wave direction was perpendicular to the barrier.

6. Rocks for Rubble Mounds:

In construction of a rubble mound the weight of the face and armor stone that will resist the forces of breaking waves is calculated from empirical equations based on wave energy, the buoyancy of the stones, and the slope of the rubble mound. The weight of the rock in tons is given by the expressions: (Reference IV G-4)

Prof. Don Edwardo do Castro April 1933

$$P = \frac{0.0218\ H^3 \gamma}{(\mathrm{Cot}\alpha + 1)^2\ (\mathrm{Cot}\alpha - 2/\gamma)^{\frac{1}{2}}\ (\gamma - 1)^3} \qquad \text{(IV G-6.1)}$$

Prof. Ramon Irabarren Cavanilles July, 1938

$$P = \frac{KH^3\gamma}{(\mathrm{Cos}\alpha - \mathrm{Sin}\alpha)^3\ (\gamma - 1)^3} \qquad \text{(IV G-6.2)}$$

where

K = 0.000 463 for natural stone
K = 0.000 593 for artificial concrete blocks
H = wave height
$\gamma$ = sp. gr. stone
$\alpha$ = angle of slope with horizontal

Mr. William J. Mathews May, 1948

$$P = \frac{6\,W\,H^2\,T}{(W-64)^3\,(\cos\alpha - 0.75\,\sin\alpha)^2} \qquad \text{(IV G-6.3)}$$

where

T = wave period, sec.
W = density of stone lbs/ft$^3$

Mr. Fred W. Rodolf

$$P = \frac{H^2 T \gamma}{600\,\tan^3(45^\circ - \alpha/2)\,(\gamma - 1)^3} \qquad \text{(IV G-6.64)}$$

Suppose concrete blocks ($\gamma$ = 2.2, W = 137 Lbs/ft$^3$) are placed on a 1:2 slope of a rubble mound and are subject to 8 ft. high breakers of 12 sec period.

$\tan\alpha = \frac{1}{2}$; $\therefore \alpha = 26^\circ\ 34'$

and

$\cot\alpha = 2.0$
$\sin\alpha = 0.4475$
$\cos\alpha = 0.8943$

By Castro's method, equation IV G-6.1

$$P = \frac{0.0218 \times 8^3 \times 2.2}{(2.0 + 1)^2\,(2.0 - \frac{2}{2.2})^{\frac{1}{2}}\,(2.2 - 1)^3} = 1.45 \text{ tons}$$

By Cavanilles' method, equation IV G-6.2

$$P = \frac{0.000593 \times 8^3 \times 2.2}{(0.8943 - 0.4475)^3\,(2.2-1)^3} = 4.33 \text{ tons}$$

By Mathews' method, equation IV G-6.3

$$P = \frac{6 \times 137 \times 8^2 \times 12}{(137 - 64)^3 \left[0.8943 - 0.75\,(0.4475)\right]^2} = 5.20 \text{ tons}$$

By Rodolf's method, equation IV G-6.4

$$P = \frac{8^2 \times 12 \times 2.2}{600\,\tan^3(45^\circ - \frac{26^\circ\ 34'}{2})\,(2.2-1)^3} = 6.90 \text{ tons}$$

These values of rock weight show a considerable range of possibilities and present an interesting difference of opinion. There is a slightly different approach to this problem in that for many structures the rock size is goverened by the limits of the hoisting and placing equipment. If there is a maximum size and weight of rock that can be successfully placed on the rubble mound and there is a known maximum wave condition or design wave then the slope of the structure is calculated. As before, there will be a range of values obtained from the above equation. Figure IV G-12 shows how the various methods compare to an actual slope of a stable rubble mound. However, this is not conclusive because if the rock is over size the shape of the structure will be unchanged by the wave action and will be stable as constructed. Thus, optimum

conditions are difficult to calculate. Using the above equations as a guide one can pick the heaviest stones necessary. The larger the armor stone possible the less will be the maintenance needed on a rubble mound breakwater provided the void areas are kept to a minimum so that core material will not be displaced.

7. The Critical Slope of a Wall for Wave Reflection(Reference IV G-2):

The critical slope of a wall is defined as the slope which is mid-way between the slope that causes a wave to be fully reflected (unbroken) and the slope that causes a wave to break without any reflection. The critical slope is different for different wave heights and periods. It is important to determine the action of the maximum wave condition and the design wave on a structure in light of the forces exerted by the waves for the two extreme conditions as previously discussed. Figure IV G-13 shows the definition of the wall slope and the symbols used below.

$$i = \frac{\ell}{d} \qquad \text{(IV G-7.1)}$$

For the critical value of the slope Ramon Iribarren Cavanilles presented the following equation

$$i = \frac{8}{T}\sqrt{\frac{H}{2g}} \quad \text{(critical)} \qquad \text{(IV G-7.2)}$$

where

$$T = \sqrt{\frac{2\pi}{g} L \operatorname{Coth} \frac{2\pi d}{L}} \qquad \text{(IV G-7.3)}$$

The Deft Laboratory indicated that if $\ell/L = 1/4$ or less the wave would be fully reflected and unbroken, and that if $\ell/L = 1/2$ or greater the wave would be broken without any reflection. Thus, for an existing structure of known slope, the wave action depends on the wave length.

If a structure is to be built, the critical slope maybe determined from the design wave and depth of water. Suppose that the wave height is 8 ft., the wave length is 500 ft. and the water depth at the structure is 30 ft. From equation IV G-7.3 we compute the wave period.

$$T = \sqrt{\frac{2\pi}{g} L \operatorname{Coth} \frac{2\pi d}{L}} = \sqrt{\frac{2\pi}{32.2} \times 500 \times \frac{1}{0.3601}} = 16.45 \text{ Sec.}$$

Now the critical slope from equation 76 is

$$i = \frac{8}{T}\sqrt{\frac{H}{2g}} = \frac{8}{16.45}\sqrt{\frac{8}{2 \times 31.2}} = 1.72$$

and from equation IV G-7.1

$$i = \frac{d}{\ell} \quad \text{given the side slope of 1:1.72}$$

and $\ell = i\,d = 1.72 \times 30 = 51.6$ Ft.

It follows that

$$\frac{\ell}{L} = \frac{51.6}{500} = \frac{1}{9.7}$$

which by the Deft Laboratory would be a fully reflected wave. It is difficult to visualize that a structure could reflect all waves of a given wave length independent of the wave height, particularly if the wave is very steep. The calculation of the critical slope of a breakwater should act somewhat as a guide to the wave action that may be expected but it should not be used to influence the calculations of the maximum expected wave force.

8. References:

IV G-1: Baynold, R. A. "Interim Report on Wave-Pressure Research"; J. Institute of Civil Engineering. No. 7; June 1939.

IV G-2: Cavanilles, Ramon Iribarren and Alano, Castro Nogales Y; "Talud Limite Entre La Rotura & La Relfexion de Los Olas"; Revista de Obras Publicas; Feb. 1950, Madrid.

IV G-3: Gaillard, D. D.; "Wave Action in Relation to Engineering Structures"; The Engineering School; Ft. Belvoir, Virginia; 1904.

IV G-4: Hickson, R. E. and Rodolf, F. W.; "Design and Constuction of Jetties"; Institute on Coastal Engineering; University Extension; University of California Oct. 11-13, 1950.

IV G-5: Hudson, R. Y. "Model Test of Portable Breakwaters for D-Day Invasion Harbors"; Civil Engineering; Vol. 15, No. 9, Sept. 1945; p. 405.

IV G-6: Larras, M. J. "The Breaking Into Foam of Waves on Vertical Jetties"; Annales des Ponts et Chaussees, Vol. 107, May 1937; pp. 643-680, Translated by Works Progress Administration, University of Minnesota.

IV G-7: Lochner, Robert; Faber, Oscar; and Penney, Willian G.; "The 'Bombardon' Floating Breakwater"; The Civil Engineer in War, Vol. 2; Docks and Harbors, 1948; Institute of Civil Engeering, London (The right of publication and of translation is reserved.)

IV G-8: Miche, M; "Mouvements Ondulations De La Mer en Profondeur Constante ou Decroissant; 1945.

IV G-9: Minikin, R. R.; "Winds, Waves and Maritime Structures"; Charles Griffin & Co. Ltd. London, 1950.

IV G-10: Morison, J. R.; "Wave Pressures on a Vertical Wall"; HE 116-298, Dec. 22, 1948; Fluid Mechanics Laboratory, Technical Report; Engineering Research Projects, University of California; Berkeley, California.

IV G-11: Office of the Division Engineer; "Study of Wave Force"; Office of the Division Engineers; Great Lakes Division, Cleveland, Ohio; Oct. 1938.

IV G-12: Sainflou, M; "Essay on Vertical Breakwaters"; Translated by Clarence H. Hotch; Prepared by Office of the Division Engineer, Great Lakes Division, Cleveland, Ohio.

IV G-13: Wiegel, R. L. "Oscillatory Waves"; (Tables) The Bulletin of the Beach Erosion Board, Special Issue No. 1; July 1, 1948.

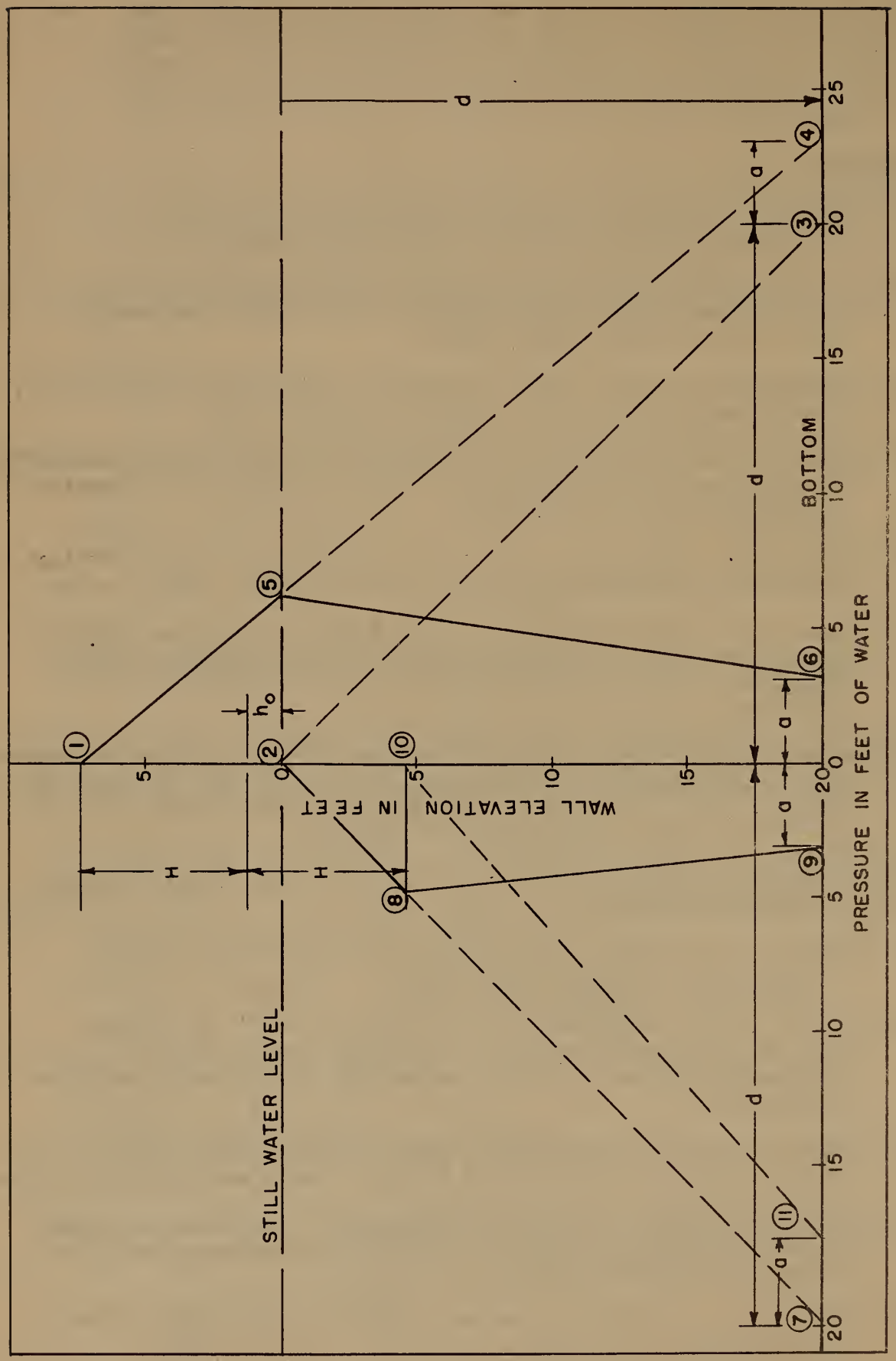



FIG. IX G-1 -- SAINFLOU'S METHOD FOR WAVE PRESSURE ON VERTICAL WALL

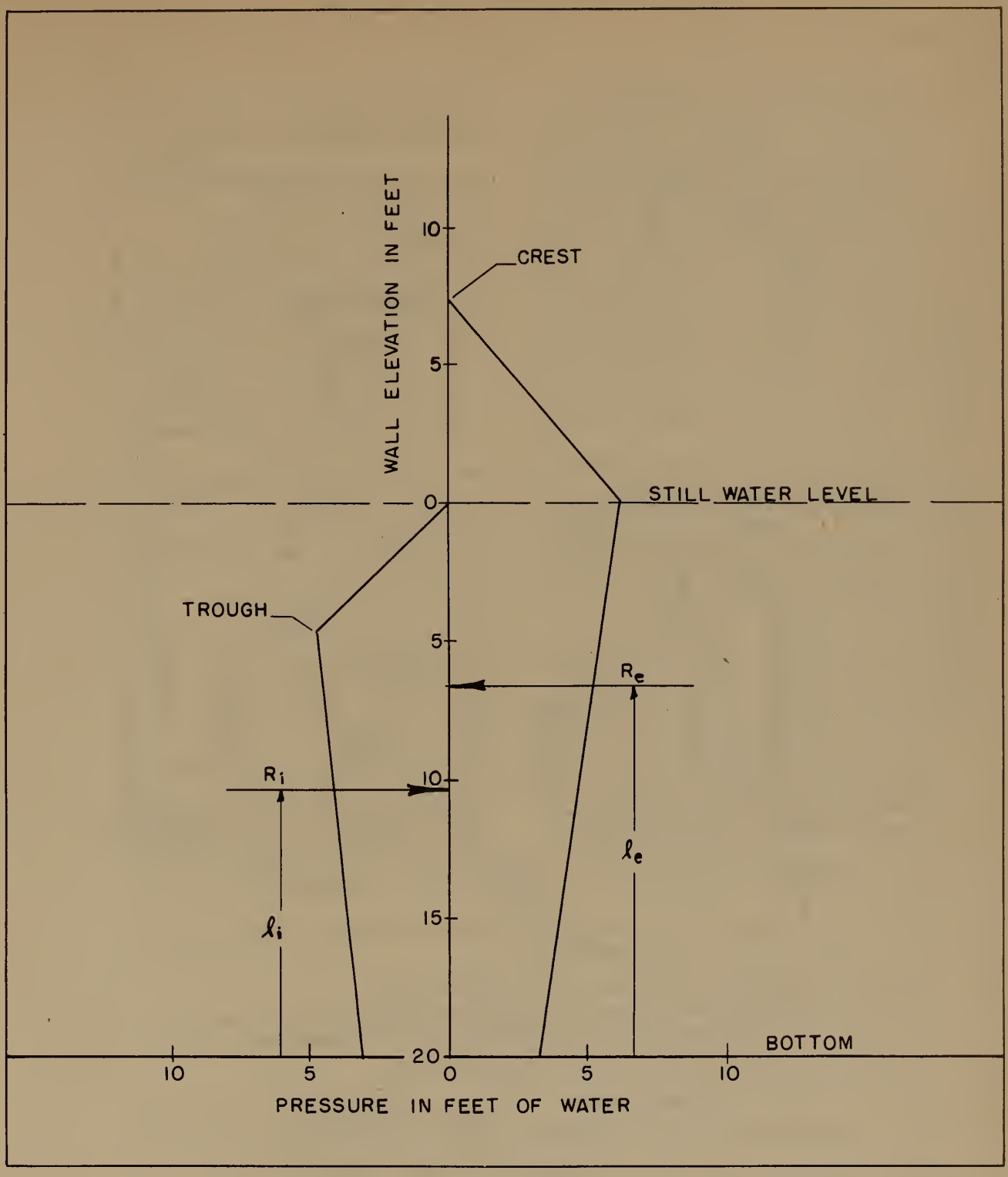
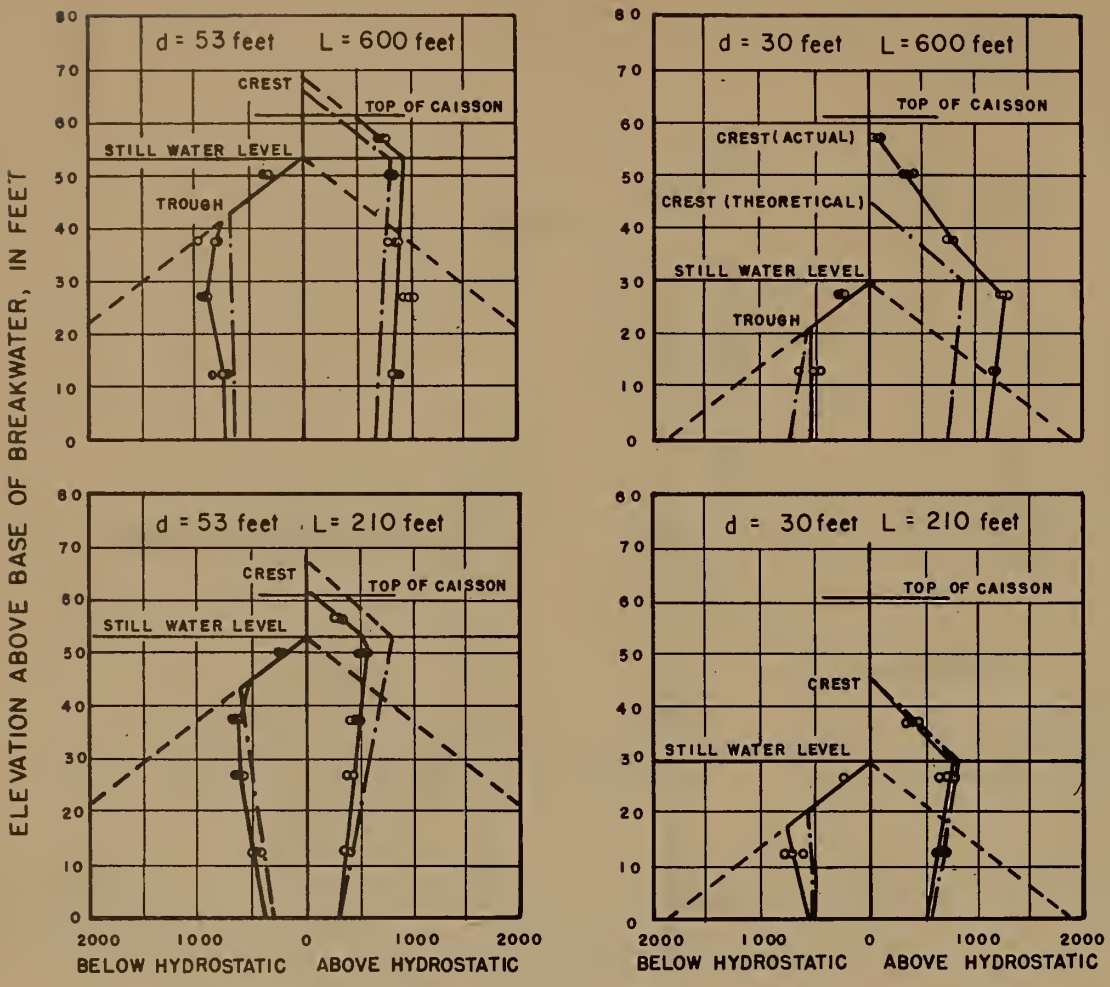



FIG. IV G-2 -- TOTAL THRUST BY SAINFLOU'S METHOD




LEGEND

– – – – – HYDROSTATIC PRESSURE (STILL WATER LEVEL)

——— MEASURED PRESSURES ON BREAKWATER ABOVE OR BELOW HYDROSTATIC PRESSURES (AVERAGE FROM THREE SEPERATE OBSERVATIONS)

—·—·— PRESSURE CALCULATED FROM SAINFLOU THEORY

d = WATER DEPTH L = WAVE LENGTH

FIG. IV G-3 - PRESSURES ON FRONT FACE OF CELLULAR CAISSON BREAKWATERS, AS DETERMINED BY MODEL STUDY.
(Waves used were equivalent to prototype waves 12 feet high)

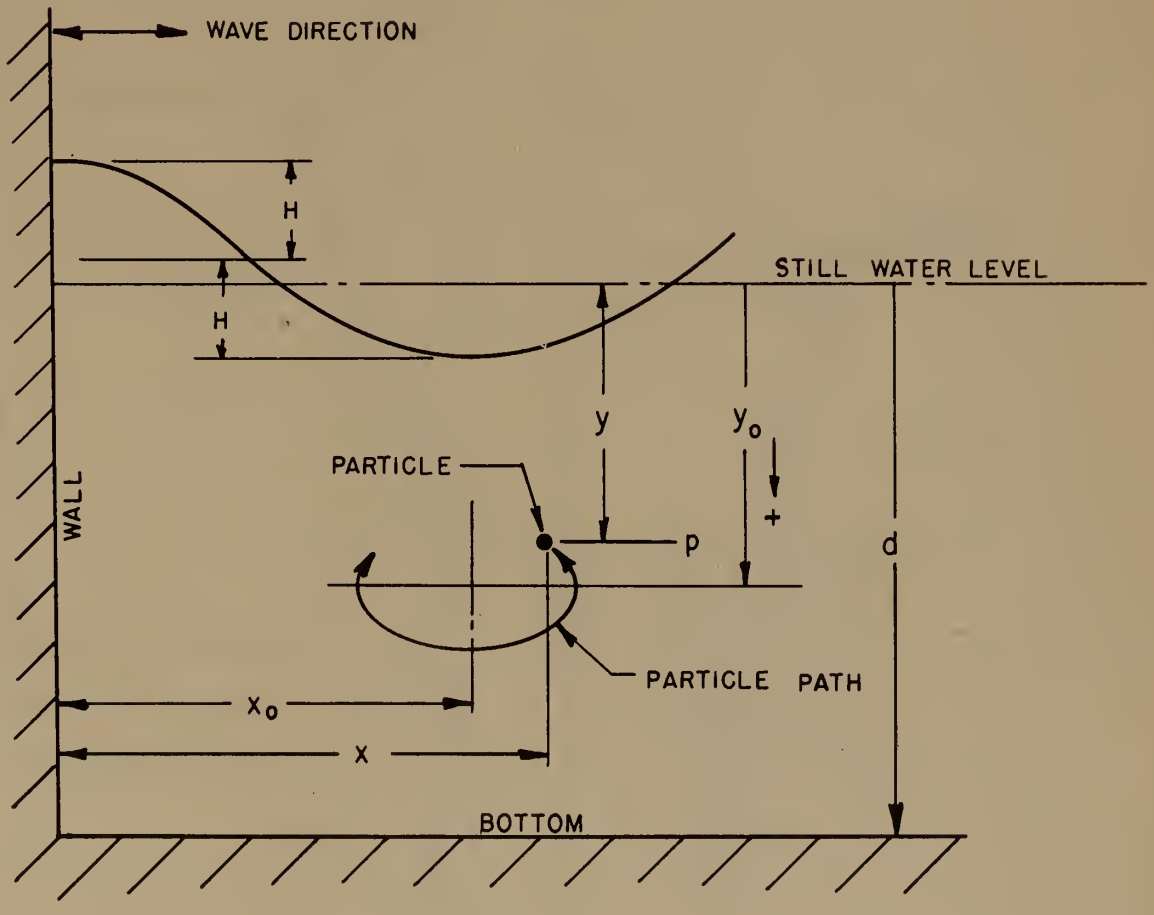



FIG. IV G-4 -- NOTATION AND SYMBOLS FOR STANDING WAVE AGAINST WALL.

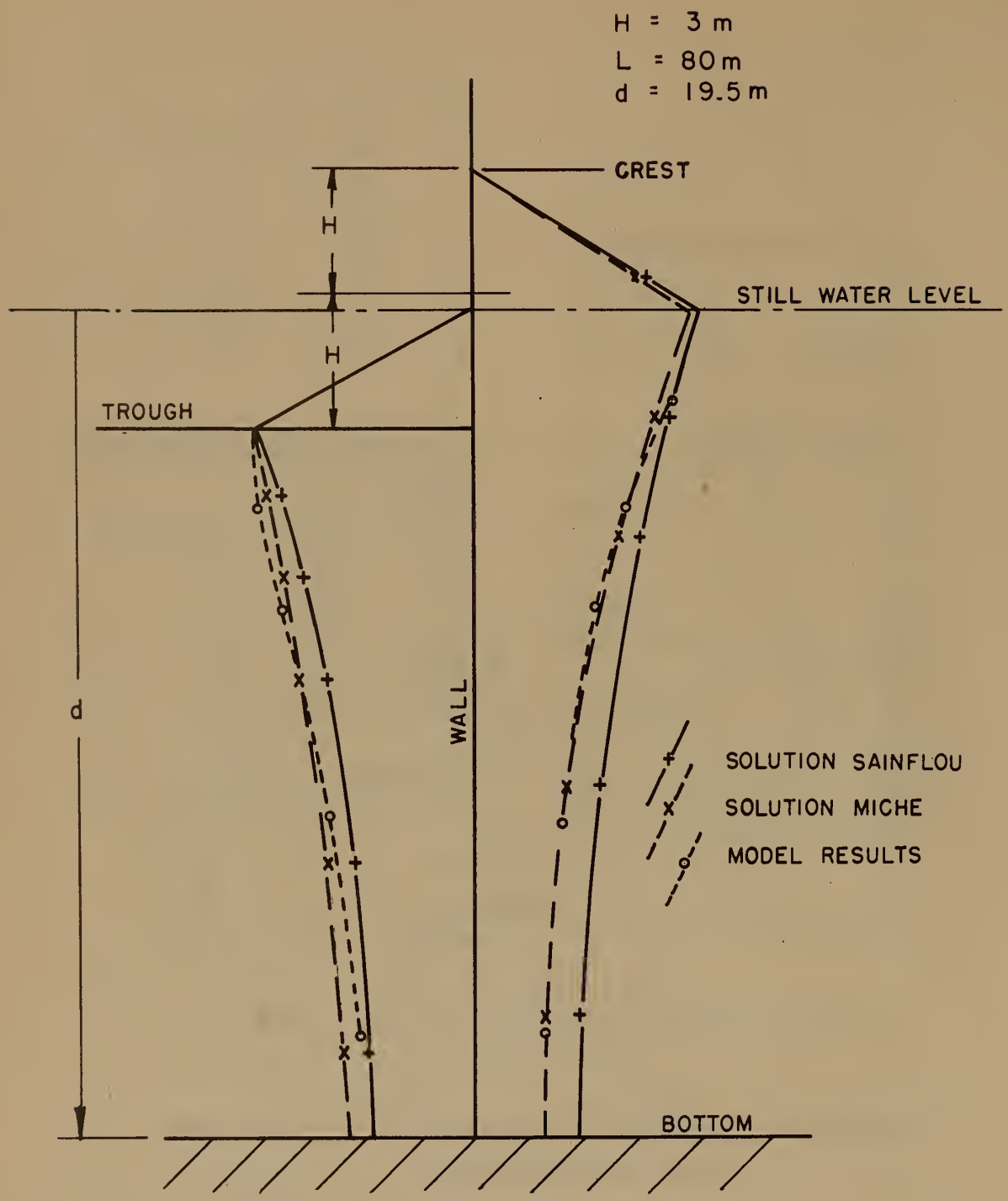



FIG. IV G-5--PRESSURE DISTRIBUTION OF STANDING WAVE ON VERTICAL WALL.

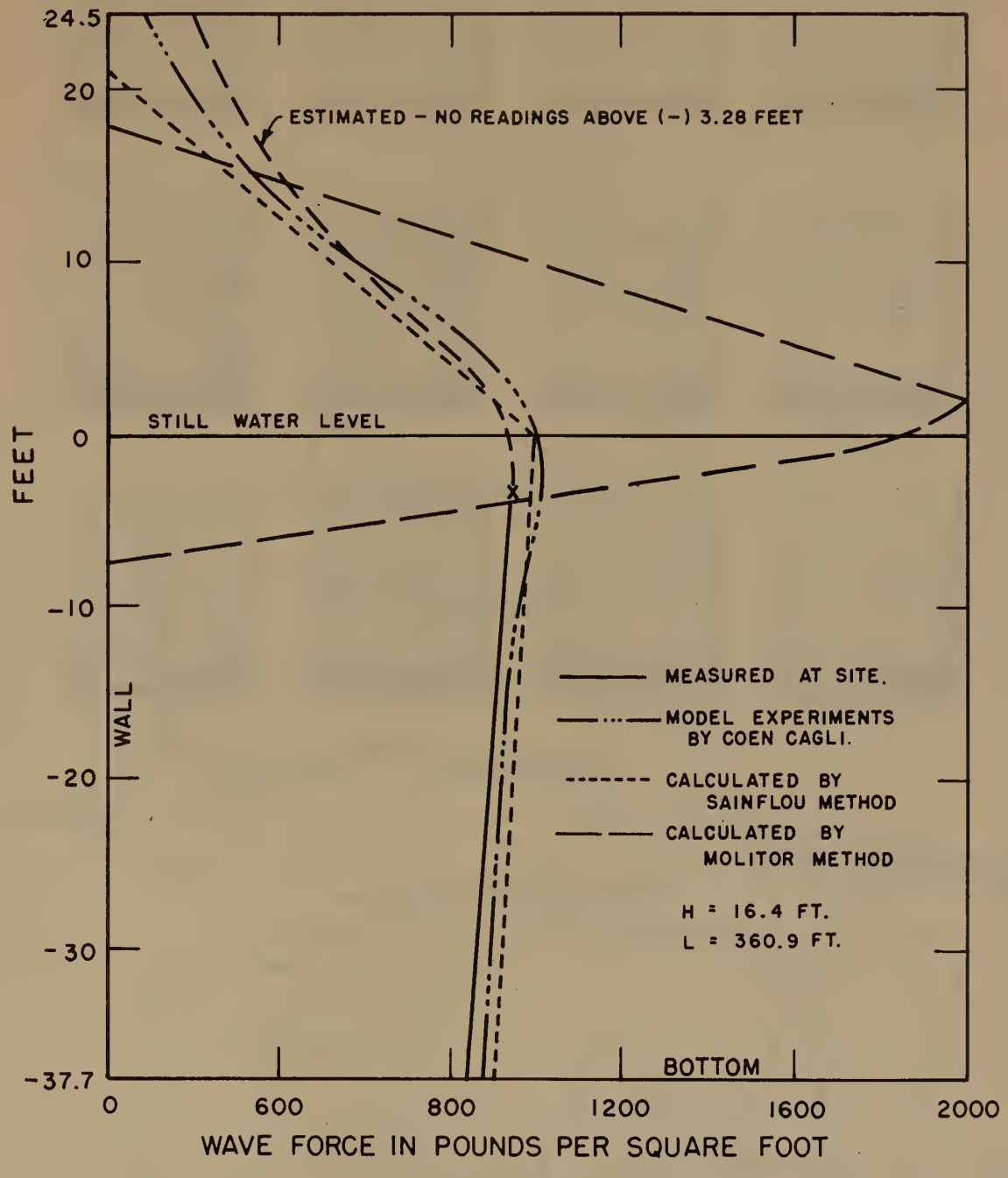
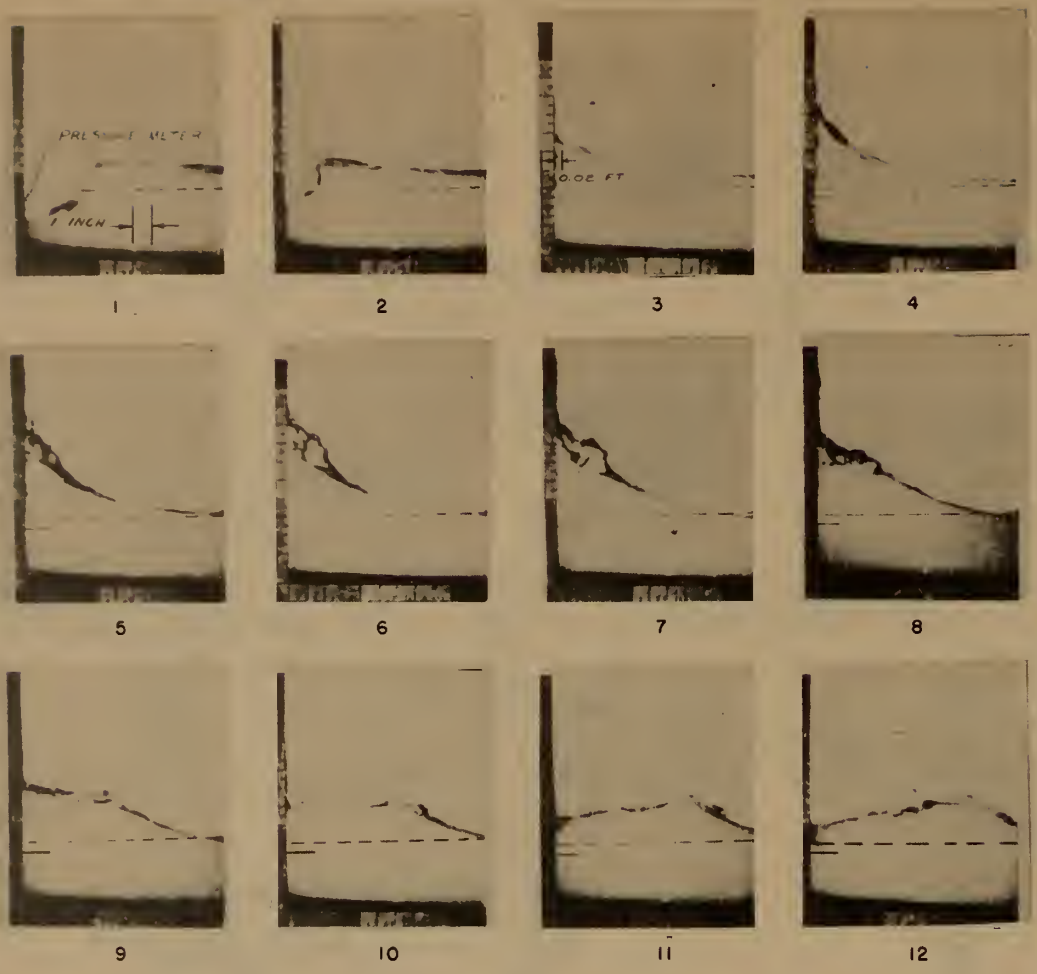



FIG. IV G-6--WAVE PRESSURES FROM MODEL STUDY AT PRINCE HUMBERT, JETTY, GENOA.



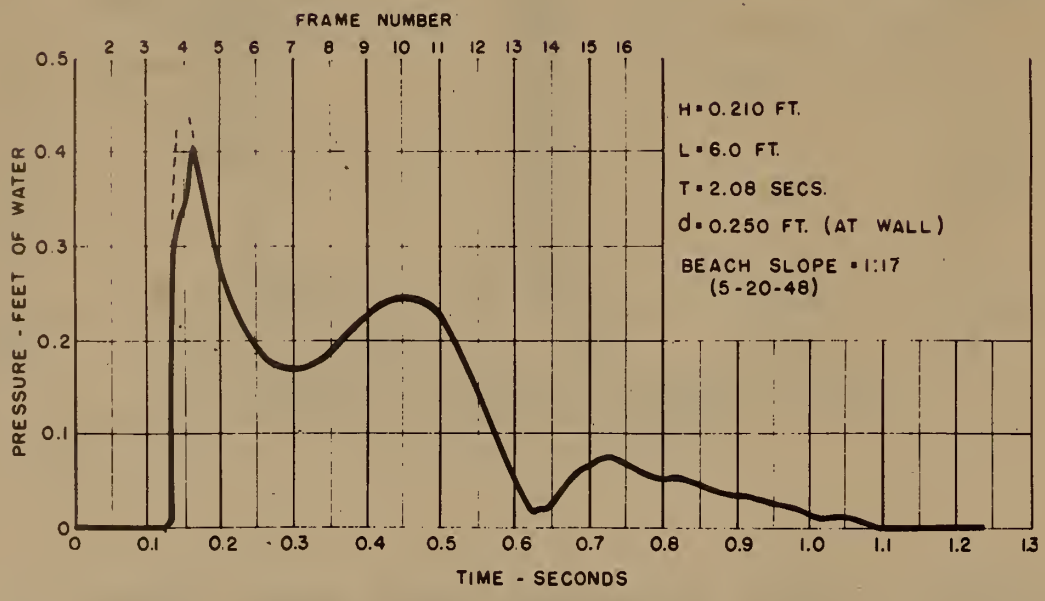




FIG. IV G-7 - WAVE PRESSURE VERSUS TIME AND POSITION
WAVE BREAKING DIRECTLY ON WALL

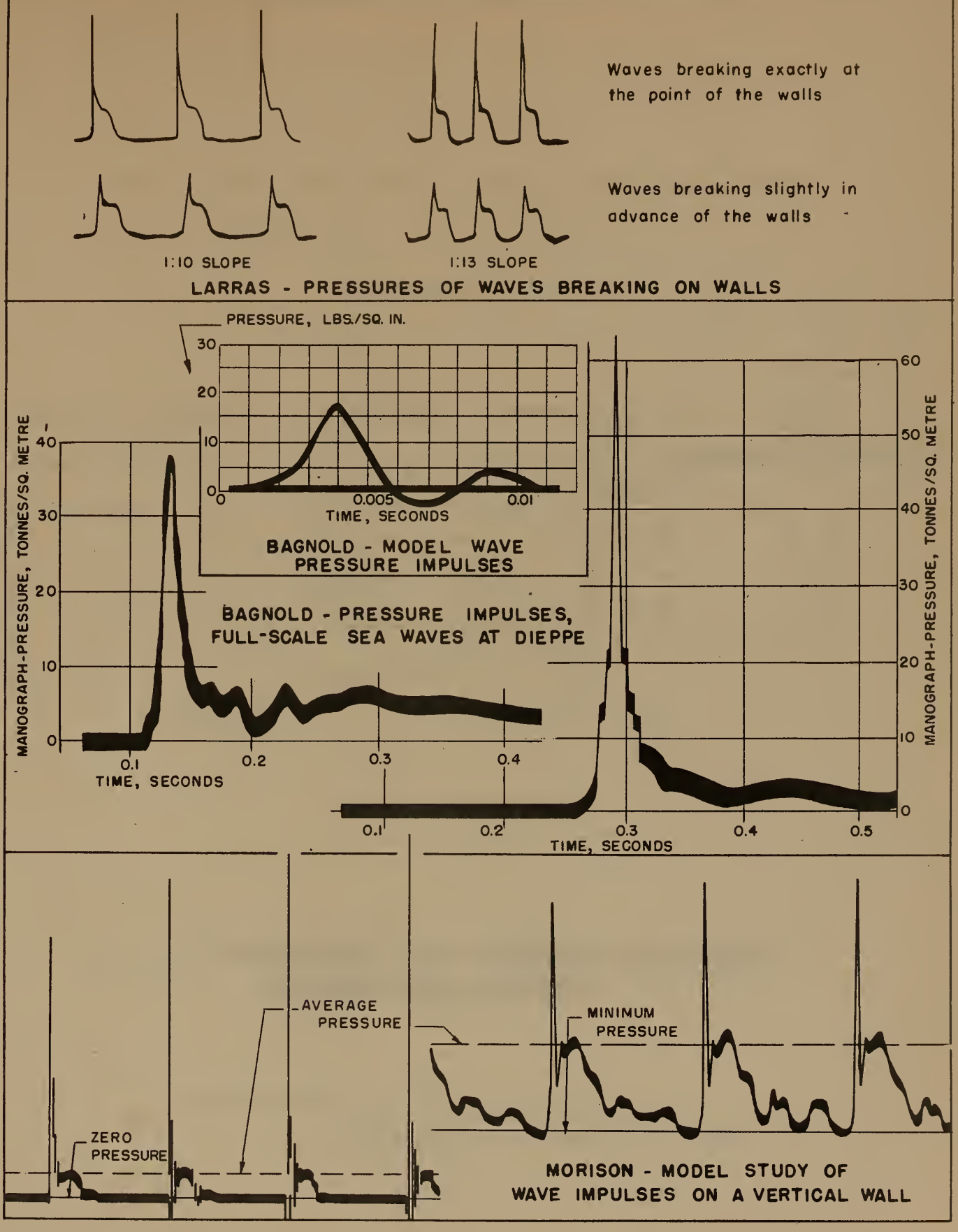



FIG. IV G-8 -- IMPACT PRESSURE OF WAVES BREAKING ON WALLS

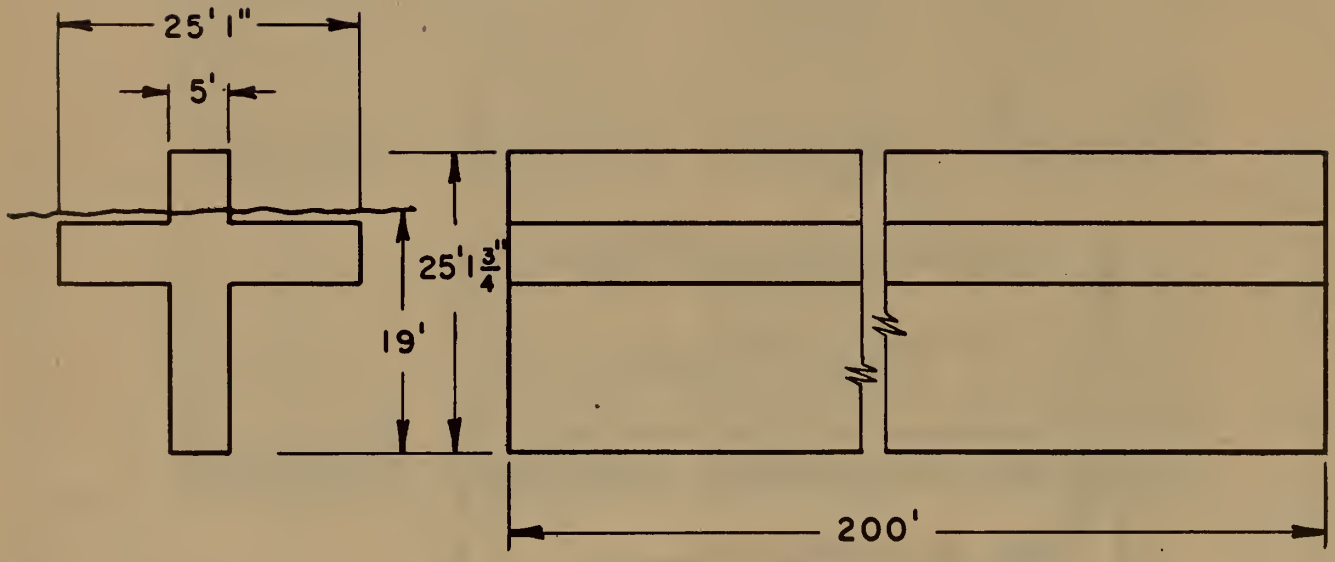



FIG. IVG-9 -- SCHEMATIC OF "BOMBARDON" FLOATING BREAKWATER.

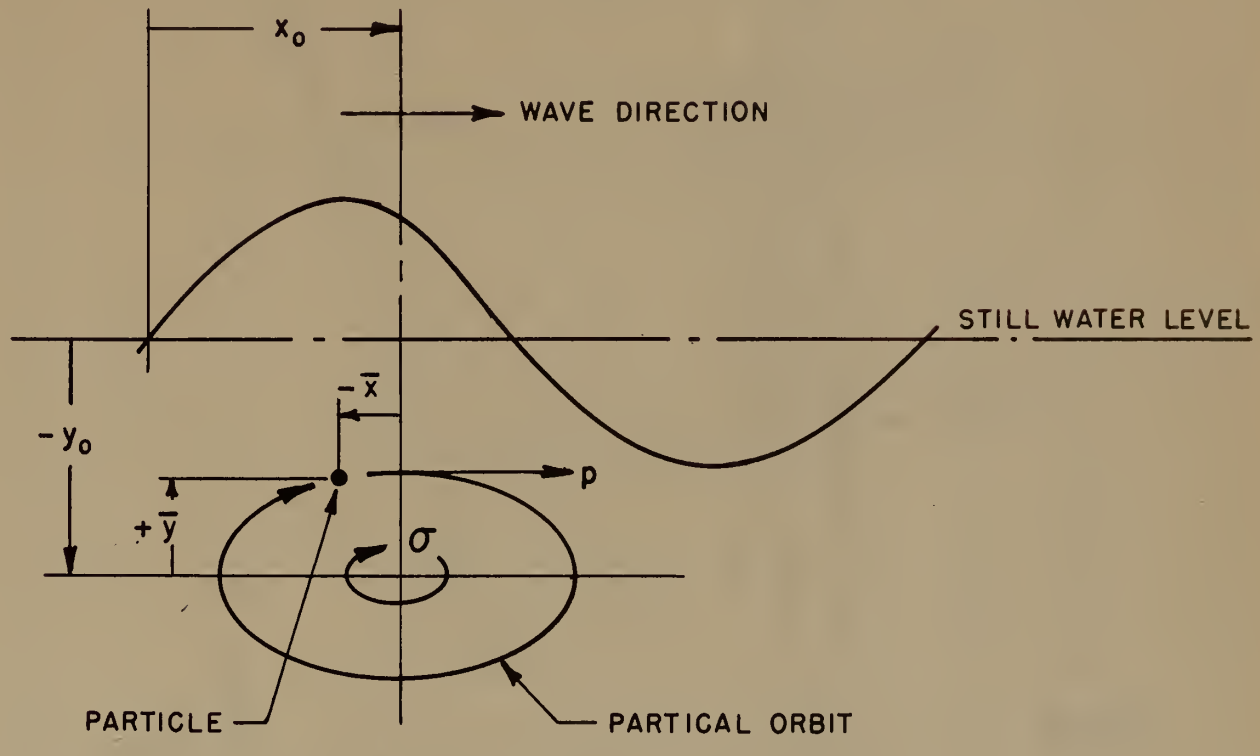



FIG. IV G-10 -- NOTATION FOR STANDING WAVE AND FLOATING BREAKWATER

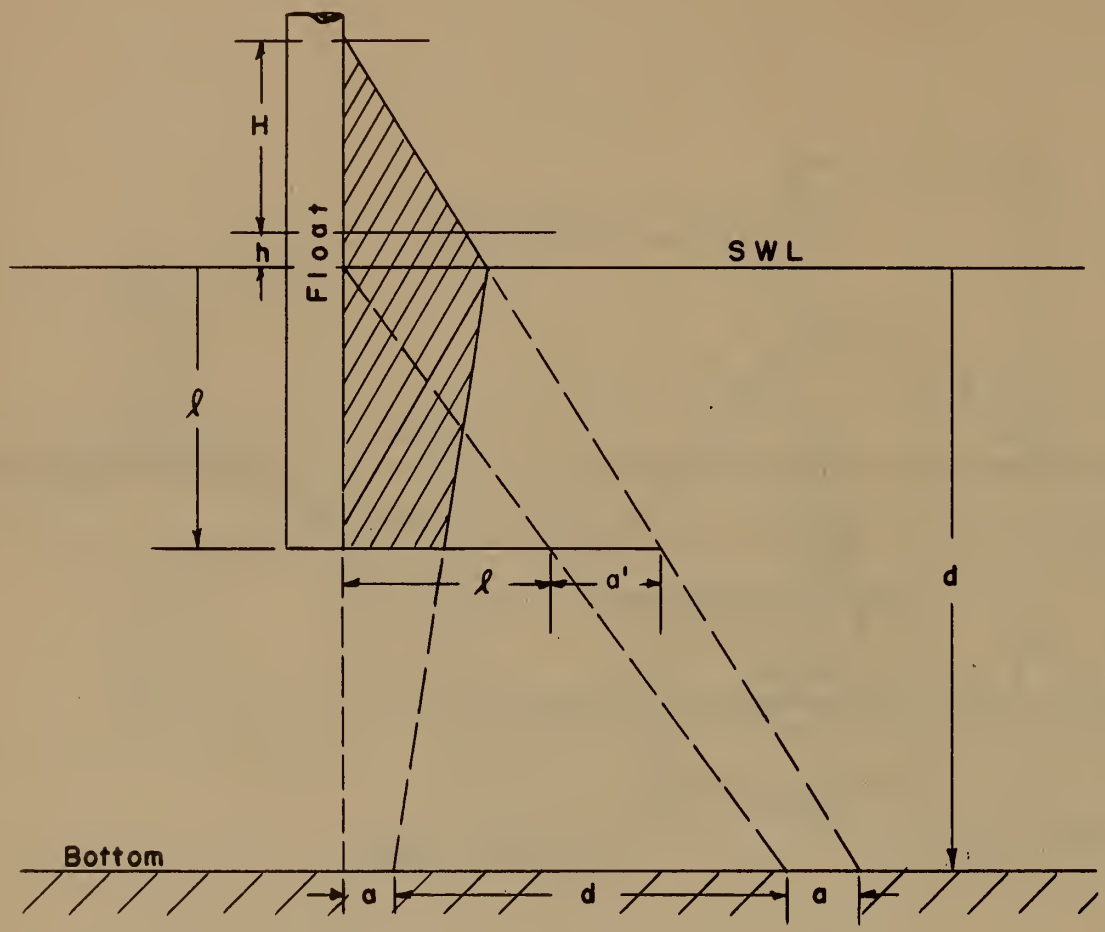



FIG. IV G-11 -- STANDING WAVE PRESSURE ON RIGID FLOAT BY SAINFLOW METHOD

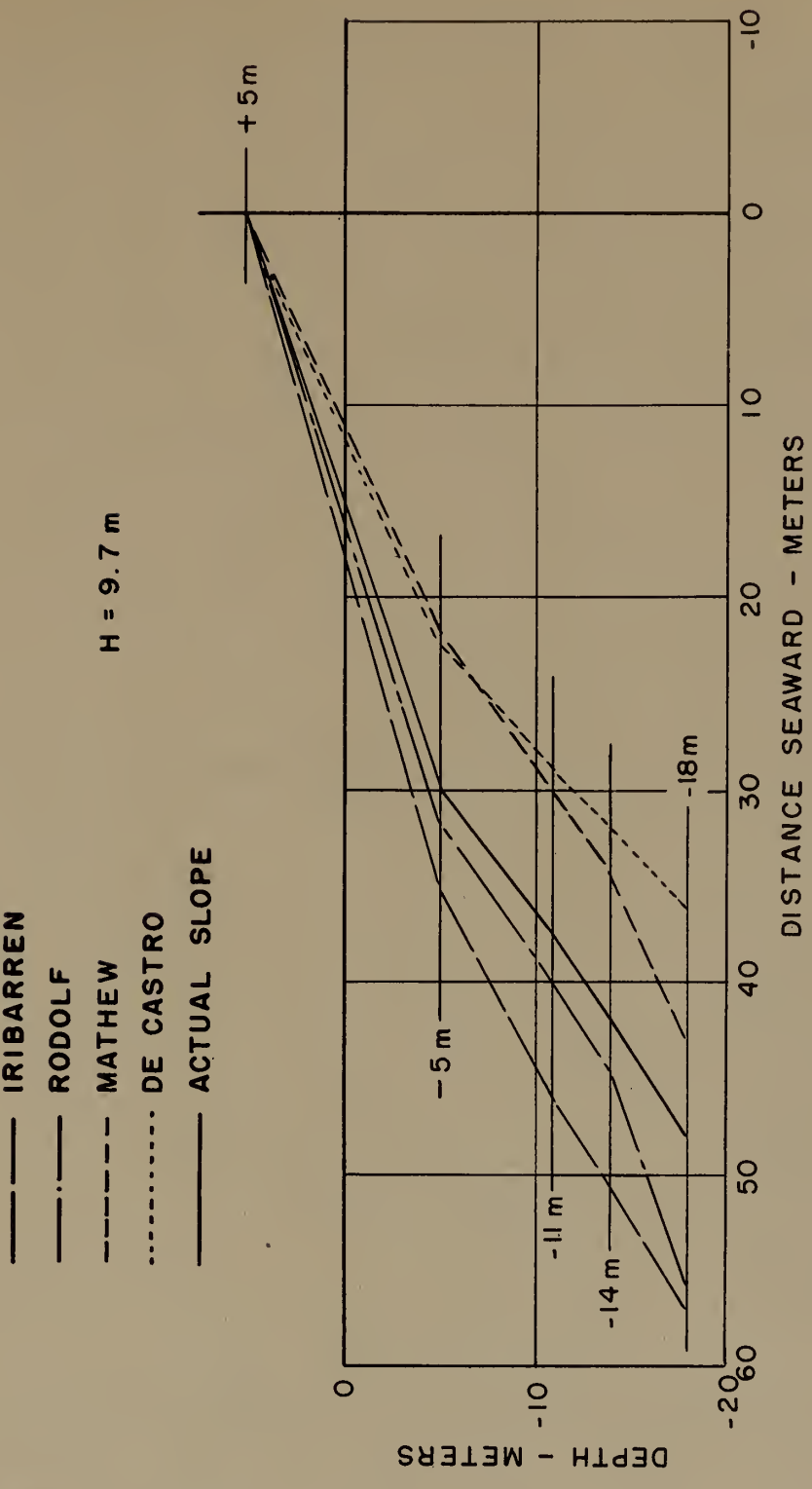



FIG. IV G-12 -- SLOPE OF ROCK DIKE AT ARGEL

from - Design and Construction of Jetties, by Hickson and Rodolf

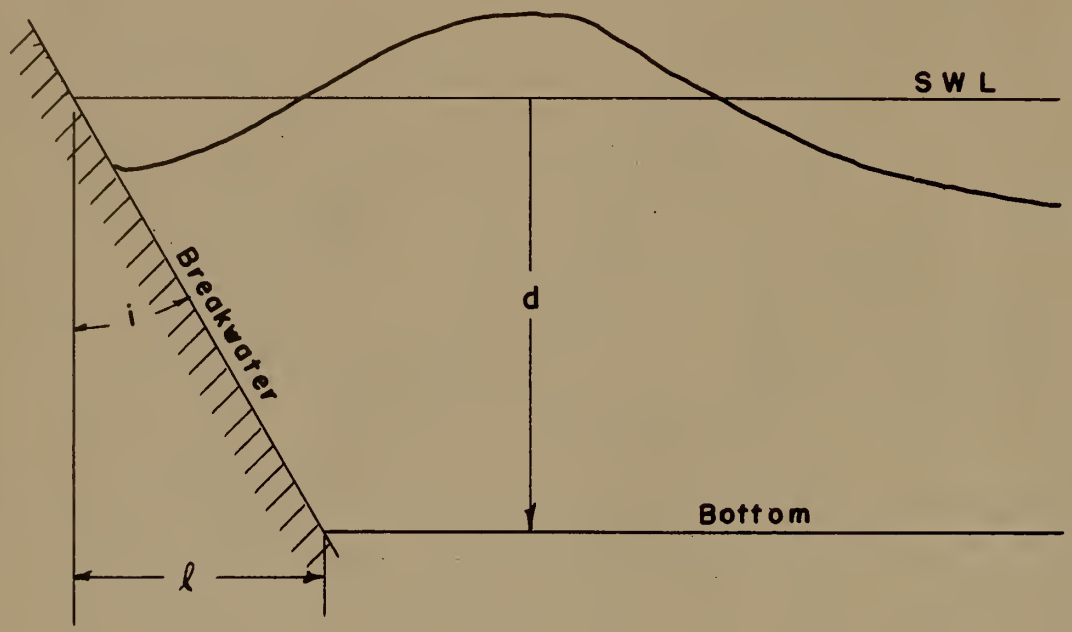



FIG. IV G-13 - INVERSE SLOPE ROTATION FOR INCLINED BREAKWATER

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# MANUAL

# OF

# AMPHIBIOUS OCEANOGRAPHY

## SECTION V

## BEACH TRAFFICABILITY AND STABILIZATION

BY

H. BOLTON SEED



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SUMMARY OF SECTION V

# BEACH TRAFFICABILITY AND STABILIZATION

BY
H. BOLTON SEED

## THE TRAFFICABILITY OF BEACHES

### Beaches and Trafficability

One of the most important factors in an amphibious operation is that of getting vehicles from the landing ship or craft across the beach and onto hard terrain. During this process, a vehicle must satisfactorily negotiate four different sections of ground:

(i) the submerged section of beach between the landing craft and the shore;

(ii) the moist section of the beach (the beach face) which is covered by water at high tide;

(iii) the dry section of the beach which is never submerged, even at high tide;

(iv) the backshore which may be sand dunes, a cliff, a slough, etc.

It will readily be appreciated that the characteristics of the four main sections of a beach will differ considerably with regard to their ability to sustain traffic. While it is possible to discuss and to some extent evaluate the trafficability of the different sections of a beach, it is evident that all four sections must be trafficable in order to satisfactorily accomplish an amphibious landing operation. Thus in assessing the trafficability of a beach, the backshore conditions must also be considered, in addition to the conditions of the beach itself.

Beach soils vary greatly in character ranging from cobbles and sand to muds. Thus all types and conditions of soils may be encountered in passing from a landing craft to the ground beyond the backshore. It is clearly desirable to be able to determine in advance of an amphibious operation whether the beach soil will or will not be trafficable. For this purpose, it is necessary to have a thorough understanding of the factors affecting the trafficability of soils and the available methods for predicting soil trafficability.

### Factors Affecting Trafficability

Trafficability may be defined as the ability of an area to sustain the motion of wheeled and/or tracked vehicles. It is evident, therefore, that the trafficability of an area depends on characteristics of both the area itself and the vehicles considered. It is not proposed to discuss, in this report, the vehicle characteristics which affect trafficability, but rather to review the area characteristics which determine whether or not motion of a vehicle is possible. Since trafficability problems on beaches are mainly concerned with areas whose trafficability has not been improved by artificial means, the area characteristics to be considered are mainly those of the natural soil deposit comprising the upper two or three feet of the area.

The trafficability of a soil is dependent on three main factors:

(i) The soil must have sufficient bearing capacity to support the vehicle.

(ii) The soil must have sufficient tractive capacity to develop the resistance, between the soil and the tire or track, required to provide the forward thrust.

(iii) The rolling resistance due to the soil and the vehicle must not be greater than the forward thrust which the vehicle can develop.

Trafficability is also dependent on other factors, but only because they cause changes in the three main factors listed above. For example, trafficability may depend on the density of traffic. On some soils, the continuous passage of traffic loads over the soil may cause the bearing capacity and traction to decrease and the rolling resistance to increase. If the rolling resistance should become equal to the thrust developed by traction between the treads and the soil, a vehicle will be immobilized. Thus a soil which was initially trafficable may become untrafficable. For this reason, an earlier report of the U. S. Corps of Engineers considered trafficability of soils for two different traffic density conditions:

(i) limited operation - when vehicle movement was less than about 25 repeated trips.

(ii) unlimited operation - when vehicle movement was greatly in excess of 25 repeated trips.

The ability of a soil to develop adequate bearing capacity depends mainly on the shear strength characteristics of the soil. These in turn depend on the soil type. If a soil has sufficient bearing capacity to support a wheel load on the surface, the deformation under the wheel is mainly due to elastic compression of the soil. However, if the surface bearing capacity is not sufficient to support the load, the wheel may sink into the soil until either the vehicle is supported mainly by flotation, or, due to the confining effect of the adjacent soil, the soil below the wheel develops the required bearing capacity. This type of settlement is due to plastic flow resulting from shear failure of the soil. If arrested, the final settlement depends on the shear strength characteristics and the unit weight of the soil.

The unit weight, together with the water content, will also indicate the danger of sinkage of vehicles due to local liquefaction of sandy soils.

In muddy areas where the soil is very weak, the first pass of a vehicle causes a rut of considerable depth and successive passes cause additional deformation and deepening of the ruts. As the depth of the ruts increases, it may eventually exceed the ground clearance of the vehicle, resulting in a decrease in traction and an increase in rolling resistance. When the rolling resistance is equal to the tractive force, the vehicle is immobilized. In such cases, the main problem of trafficability is either that of floating the vehicle on the soil or of estimating the number of passes before failure will occur.

The tractive force between the tire or track of a vehicle and the underlying soil is partly developed by the provision of treads (on tires) or grousers (on tracks) which dig into the upper layer and develop thrust due to the passive resistance of the soil to deformation. The maximum passive resistance of a soil is also a function of its shear strength characteristics.

Alternatively, if the track or tire is not able to dig into the soil, the tractive force is developed by friction between the wheel and the soil. Even a very hard material may have a low coefficient of friction, a condition referred to as slipperiness. This condition may also occur when a hard soil is overlain by a thin layer of very soft weak soil which causes an apparent lubricating effect so that little traction is developed.

When a wheel or track sinks into a soil, it is necessary for the vehicle to push soil out of the way in order to move forward. This resistance of the soil is an important part of the rolling resistance. If the soil resisting forward movement of the wheel is weak, it can be pushed away relatively easily, but if it is strong, it can withstand considerable pressure. Thus the rolling resistance is a function of the shear strength of the soil.

Other factors affecting rolling resistance are the slope of the surface, the vehicle characteristics and the stickiness of the soil. Under certain conditions, some types of soil become sticky and adhere to the tracks and tires of vehicles, accumulating on the track rollers or beneath the fenders or carriage and thus increasing the rolling resistance. This accumulation of sticky soil is termed mud-packing.

Hence, the trafficability of a soil depends on three main factors -- bearing capacity, traction capacity and rolling resistance. These factors in turn depend on the vehicle characteristics, the traffic density and the slope of the ground, in addition to the following soil characteristics:

| | |
|---|---|
| Soil type | Friction |
| Shear strength | Stickiness |
| Unit weight | Slipperiness |

With the exception of soil type, all of the above soil characteristics depend on the water content of the soil. The water content is, therefore, an especially important factor in determining soil trafficability.

In order to accurately predict the trafficability of a soil, it is first necessary to evaluate the soil characteristics listed above. The methods used to evaluate these characteristics are summarized below:

Soil type: The U. S. Corps of Engineers has proposed that a modified form of the Casagrande Airfield Soil Classification system be used to classify soils with respect to trafficability. This system is well-suited for trafficability classification purposes. Soils are divided into two main types -- coarse-grained cohesionless soils, such as sands and gravels, and fine-grained soils, such as silts and clays. Sands and gravels are further classified on the basis of grain size distribution and silts and clays on the basis of their plasticity characteristics as determined by their liquid and plastic limits.

The modification proposed by the Corps of Engineers is to classify fine-grained soils on the basis of plasticity index rather than liquid limit. This is suggested because of the apparent correlation between stickiness and plasticity index. However, if some other more satisfactory method of measuring stickiness were developed, this modification would be unnecessary.

Shear Strength: The shear strength characteristics of the soil may be determined by laboratory or field shear tests, together with measurements of the water content of the soil in situ. An instrument - the "soil truss," has been developed at Port Hueneme for measuring the shear strength characteristics of a soil in situ.

Shear strength characteristics can also be evaluated by empirical penetration tests, in which the resistance of the soil to penetration by a standard instrument is measured. Various types of penetration devices have been developed, but the instrument which has been most widely used is the cone penetrometer developed at the Waterways Experiment Station, Vicksburg. It consists of a 30" cone with a 1 sq. in. projected area mounted on a 5/8" diameter shaft. Load is registered by a proving ring. The cone is pushed into the ground and the point at which movement begins is determined by "feel". The load in pounds at which movement first occurs is called the cone index of the soil. The cone index is a quantitative index of the strength of the soil.

Unit Weight: The unit weight of a soil can be determined, when required, by field density measurements.

Friction: The coefficient of friction between a tire and a soil may be determined by measuring the drawbar pull which can be developed by a vehicle before slippage of the wheels occurs. An attempt has been made by the Corps of Engineers to correlate the ratio of drawbar pull/weight of vehicle in such tests with the cone index of the soil. Some difficulty was experienced in obtaining a satisfactory correlation, and further data is required before any definite conclusions can be drawn. It would be expected that the coefficient of friction would also depend on the soil type and the water content.

Stickiness: Investigations by the Corps of Engineers have indicated that stickiness is related to the soil type and the water content. A modified form of Atterberg's stickiness test was used to determine when the soil was in the sticky range. A stainless steel spatula was drawn over the surface of a soil sample at various water contents. If no soil adhered to the spatula, the stickiness was noted as "none"; if some soil adhered but areas of the spatula were still exposed, the stickiness was noted as "some"; if the soil completely covered the face of the spatula, the stickiness was noted as "complete." In this manner, the ranges of water contents for "some" and "complete" stickiness were determined. These ranges were then related to the soil type by expressing them as ranges of water/plasticity ratio. It was found that there was reasonable consistency in the water plasticity ratio at which stickiness was first noted, the values ranging between

44 and 71% for silty and sandy clays and between 26 and 40% for clays. There was little consistency in the water/plasticity ratios at which "some" stickiness occurred.

However, the spatula test was not too successful in predicting whether or not the soil would stick to a vehicle. When the reading in the spatula test was "none" or "some" no stickiness of soil to vehicle occurred in trafficability tests; when the spatula reading was "complete," in some cases stickiness of the soil to a vehicle occurred and in other cases no stickiness was observed.

Another disadvantage of the spatula test is that it does not indicate the degree of stickiness of a soil. The only means for determining the relative stickiness of soils is the approximate correlation, observed in the tests described above and in other fields, that the higher the plasticity index, the stickier the soil.

Further work on the stickiness characteristics of soils, with the possible development of a new test for measuring stickiness, is required before the effects of this factor can be satisfactorily included in an analysis of soil trafficability.

Slipperiness: An attempt has also been made by the Corps of Engineers to correlate slipperiness with water content and soil type by means of the water/plasticity ratio. Again, however, the correlation was not very definite. For rubber tires, the data indicate that practically any water content above the plastic limit is too slippery for a vehicle to operate and that for tires with traction devices the slippery range is just beyond the sticky range.

## Methods of Predicting the Trafficability of Soils

Two methods have been used to predict the trafficability of soils: the rational approach to the problem and the empirical approach.

### 1. Rational approach to the prediction of trafficability.

In this method the shear strength characteristics and the unit weight of the soil are determined and are then used to evaluate the behavior of the soil under traffic loads. An apparently satisfactorily instrument, the soil truss, has been developed to measure the strength characteristics of a soil in the field.

However, an adequate theory for the mechanics of soil under wheel or track loads is required. A satisfactory formula for determining the tractive force developed by a tracked vehicle is available. Similar formulas for the tractive force developed by wheeled vehicles both with and without traction devices are required, together with a theory for evaluating the depth of sinkage and the rolling resistance. In addition, reliable methods of evaluating slipperiness and stickiness and consideration of these factors are required in making a complete analysis. From these results, the drawbar pull developed by a vehicle, which is the index of trafficability, could be obtained.

A start on this work has been made at Port Hueneme, but, at the present time, the method has not been developed sufficiently for trafficability prediction.

2. Empirical approach to the prediction of trafficability.

Owing to the difficulty of analyzing the stresses in a soil mass under a wheel load and thus determining the sinkage and rolling resistance, various attempts have been made to predict trafficability by an empirical correlation of vehicle performance with the shear strength of the soil. In these correlations, the shear strength has not been measured directly, but some indicator test has been used which is itself related to shear strength. In order to simplify the procedure as much as possible, the indicator test has usually taken the form of some type of penetration device. In the majority of investigations, the U. S. Corps of Engineers' cone penetrometer has been used. The resistance to penetration of this instrument is called the cone index.

In the empirical method of predicting soil trafficability, the cone index is used as an indication of the bearing capacity and traction capacity of the soil. The cone index of different soils is correlated with the trafficability of the soils and the correlation is then used to predict the trafficability of other areas. Investigations have shown that this method is apparently satisfactory for fine-grained soils. With the aid of a correlation chart and an evaluation of the stickiness of the soil under consideration, a prediction of the soil trafficability can be made.

However, it has not been found possible to establish a correlation between cone index and trafficability for coarse-grained soils. This difficulty is probably due to the unsuitability of the cone penetrometer for determining the bearing capacity of cohesionless soils. For these types of soil the bearing capacity is greatly affected by the confining pressures on the soil. Under vehicle loads confinement is produced by the width of the tires or tracks, but no confinement is used in the cone penetrometer test.

Further investigations of both the rational and the empirical methods of determining soil trafficability are required before a satisfactory method or methods for all types of soils can be developed.

General Conclusions from Trafficability Investigations.

From the investigations conducted to date, the following general conclusions can be drawn:

1. Fine-grained soils:

These soils present trafficability problems when the water content is sufficiently high (between liquid and plastic limits) that the soils are termed "muds." Correlation tables have been developed for predicting the trafficability of muds within broad limits. For each type of vehicle there is a certain cone index

requirement for vehicle mobility. The requirement depends on the depth of the supporting layer, the strength of the soil above this layer, the slipperiness and the stickiness. It has been found that the Weasel has the best trafficability characteristics of any vehicle investigated; it can operate on extremely soft muds having a cone index as low as 10, but is very susceptible to immobilization by soil jamming in the track mechanism.

2. Coarse-grained soils.

With regard to the trafficability of sandy beaches, it has been found that the flatter the beach, the better is the trafficability. It has also been shown that these soils have sufficient bearing capacity and tractive properties for trafficability provided the soils are confined. This confinement can often be achieved by the use of wide tracks or large low pressure tires.

The trafficability of wet sand is also better than that of dry sand. From this point of view it is desirable that amphibious craft should discharge their vehicle cargo during low tides, so that use may be made of the moist section of the beach face which is exposed during low tides. Further, landings made shortly after a heavy rain will encounter better trafficability on the normally dry berm-sand.

On many beaches, tractors can operate while wheeled vehicles cannot. However, where both types of vehicles can operate, tractors tend to disturb the surface and make subsequent operations with wheeled vehicles very difficult if not impossible on all but very flat beaches. During landing operations on all steep and most moderate beaches, it is desirable that tracked vehicles should be present to pull wheeled vehicles immediately away from landing craft as they unload.

On sandy beaches, as well as muds, the Weasel has been found to have the best trafficability characteristics of any vehicle investigated. It would be advantageous, therefore, to equip a Weasel for use by the Beach Master (or any other person who needs to travel a beach considerably during a landing operation) rather than a wheeled vehicle.

## BEACH SOIL STABILIZATION

### Introduction:

Although some beaches are able to sustain the motion of wheeled and tracked vehicles, in many cases the soil is not sufficiently stable to enable traffic to pass from landing craft to backshore. For example, while tracked vehicles can operate on most sandy beaches, wheeled vehicles are soon bogged down on all but the flattest beaches. On muddy areas both wheeled and tracked vehicles have great difficulty in operating, as for example, in the landings at Inchon, Korea. Thus, in amphibious warfare landing operations may be slowed considerably or even entirely prevented due to the nature of the beach material.

When the beach material is non-trafficable or has very limited trafficability, vehicles may be gotten ashore by means of pontoon causeways, landing mats or other types of temporary roadways. Alternative solutions to the problem of enabling vehicles to operate on beaches are:

(i) the design and use of equipment that can operate on any beach;

(ii) the improvement of the properties of the beach material to render it trafficable for ordinary types of vehicles;

(iii) a combined attack by improving the design of vehicles and improving the stability of beach soils.

The process of improving the properties of soils by physical and/or chemical means in order to increase their stability is termed "soil stabilization". Many methods of stabilizing soils have been proposed and used with varying degrees of success. In order for a stabilization process to be satisfactory for use on beaches during an amphibious operation it must not only cause a considerable increase in soil stability but also be rapid and relatively simple in application and should desirably be cheap in cost.

Although soil stabilization is listed separately above as a means of improving the trafficability of a beach, it cannot be entirely dissociated from the problem of equipment design since

(i) the equipment employed in the stabilization procedures must be able to move freely on the beach

(ii) improved equipment design will lower the stability requirements of the beach soil and lead to easier or more economical stabilizing procedures.

The question inevitably arises as to what degree of stability is required of a beach material and how extensive should be the stabilized zones. The requirements of the Marine Corps in these respects have been concisely stated. In the initial stages of a landing, a $2\frac{1}{2}$ ton truck should be able to cross the beach carrying a load of 19,000 pounds. In the later stages heavier vehicles must be put ashore. It is estimated that about 10,000 linear feet of stabilized roadway might be required for a unit landing operation.

### Methods of Soil Stabilization

Three general methods are available for increasing the stability of soils:

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(i) Densification of the soil;

(ii) Addition of cementing agents to the soil;

(iii) Elimination of excess water from the voids in the soil and the prevention of excess moisture accumulation.

Based on these methods, the following stabilization procedures are in more or less common use at the present time.

1. Densification:

The most stable soils contain enough coarse particles to form a connecting internal skeleton and sufficient fine particles to fill the voids between the coarser particles. This requires that the soil should be well-graded with grains ranging in size from coarse to fine. Thus, the stability of a soil may be increased by granulometric improvement; that is, by the addition of filler and binder soil to coarse-grained material or of coarse-grained material to silt and clay soils.

Densification may also be attained by compaction of a soil by some type of equipment. Tractors and vibrating equipment are most satisfactory for cohesionless soils and sheepsfoot rollers for cohesive soils. Vibration is the best method of compacting beach sands and a comprehensive study of the compaction of cohesionless soils by vibration with the object of providing criteria for the design of improved compacting equipment has recently been completed at the California Institute of Technology.

2. Addition of Cementing Agents:

The cementing agents most commonly used for soil stabilization are Portland cement, lime, bitumens and resins. In addition, certain compounds, which produce cementing materials by chemical reaction are also used. Typical examples of this are the use of sodium silicate in calcareous, aluminous or ferriferous soils or the use of this chemical together with Ca-, Fe-, or Al-salts or with aluminates or ferrates. For example, sodium silicate mixed into soil and then reacted with calcium chloride forms a gel which fills the voids in the soil and binds the soil grains together.

The most satisfactory procedure for stabilizing beach sands available at the present time is a resinous method of stabilization, using anilene-furfural resin, developed by Dr. Hans J. Winterkorn at Princeton University. A satisfactory method of chemical stabilization based on the polymerization of calcium acrylate by means of suitable catalysts and suitable for a wide range of soil types has also been developed at the Massachusetts Institute of Technology. These methods are described in a later section of this report.

3. Fusion or Thermal Stabilization:

This method involves either:

(i) The baking of cohesive soils in situ to make a passable surface or in kilns to be subsequently employed as artificial aggregate for granular stabilization.

or (ii) The use of thermite and fluxing agents to fuse the grains of beach sand together. This method has been investigated by the Bureau of Yards and Docks under the direction of Mr. Palmer and the results of the investigation are described later.



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4. Electro-chemical Hardening:

This stabilizing process involves the following phenomena:

(i) water removal by electro-osmosis;

(ii) alteration of the surface-chemical properties, especially the water-affinity of cohesive soils; and possibly

(iii) destruction of some of the soil constituents with the resultant formation of cementing substances.

5. Water Removal:

This method may involve either the removal of excess gravity water by ditching, wellpointing, subsurface drains, etc. or of non-gravity capillary and film water by evaporation, electro-osinotic movement, thermo-osmotic movement or internal drying.

Soil Characteristics Affecting the Stabilization of Beach Sands:

The following features are typical of beach sands from many parts of the world:

(i) the sand grains are of rather uniform size and relatively rounded;

(ii) the sand has a variable water content;

(iii) salinity and alkalinity of the water.

These characteristics have important effects on the stabilization of beach sands. Because of the uniform size of the grains the sands cannot be densified so much as well-graded materials and an increase in stability could be achieved by granulometric improvement. In consequence of the absence of finer material to fill the voids between the grains, beach sands require the addition of from 7% to 20% or more of material passing a No. 200 sieve to make them susceptible to bituminous stabilization by means of economical amounts of Portland cement.

The presence of water in the voids of a sand results in an increase in strength due to "apparent cohesion". When water menisci are formed at the points of contact between grains or at the surface of the soils, the surface tension in the menisci causes the grains to be pulled together and results in the development of increased frictional resistance. This strength due to the menisci or apparent cohesion is lost if the menisci are destroyed by drying or submerging the sand. Apparent cohesion is partly responsible for the fact that the moist section of a sandy beach is much more trafficable than the dry or submerged sections and is also responsible for the improved trafficability of the dry berm after it has been sprayed with water or after a heavy rain.

A number of stabilization methods are definitely affected by the salinity and/or alkalinity of the soil material and in addition, sea-water is often the only water available for construction. It is important therefore to ascertain the influence of salinity and alkalinity on potential stabilizing materials.

Variability of Beaches:

Although most beaches are composed of sandy materials, beach soils may vary widely in type and composition. The grain size of cohesionless beach materials may range from fine sand to gravels (up to the size of cobbles and boulders found on shingle beaches), the material may be well-graded or poorly-graded and may

consist predominantly of quartz, or carbonate or volcanic debris. Landings may also have to be made on beaches composed of fine-grained soils such as silts and clays. When these types of soils have a relatively high water content they become soft and plastic and are termed "muds". Mud flats present serious problems of trafficability and their characteristics are very different from those of sandy beaches.

Thus, while stabilization processes are especially required for sandy beaches it is also necessary that processes should be developed for all types of soils and conditions.

It is clearly improbable that a single soil stabilization process will prove most suitable for all types of beach soils and conditions. In any given case, the selection of the stabilization procedure to be used will depend on:

(i) the time available for stabilizing the soil;

(ii) the amount of ship space available to transport the stabilizing material and equipment;

(iii) the local availability of stabilizing materials;

(iv) specific beach properties.

Development of Practical Methods of Beach Soil Stabilization:

In the Spring of 1947 the Bureau of Yards and Docks arranged for a survey and analysis to be made of the theory and practice of soil stabilization. As a result of this survey and preliminary field tests, it was recommended that investigations of the thermite-fusion method of soil stabilization and the anilene-furfural method of resinous soil stabilization should be continued with the object of developing a practical field method for the stabilization of beach sands.

In addition, a soil solidification project sponsored by the Engineer Research and Development Laboratories at Fort Belvoir, Virginia, has been conducted at the Massachusetts Institute of Technology. After testing many materials, the best solidifier developed by this project is calcium acrylate used in conjunction with a suitable catalyst.

Present developments in the anilene-furfural, thermite-fusion and calcium acrylate methods of soil stabilization are summarized below:

1. Anilene-furfural Method of Beach Sand Stabilization:

This method of stabilization is based on the ability of two molecules of anilene to react with one molecule of furfural to produce a synthetic resin which has very strong adhesion to the sand minerals and which has good cohesive properties. The liquids are added separately to the sand, mixed intimately and the mixture is them compacted. The rate of reaction can be controlled by catalysts, the more successful ones being $FeCl_3$ and pentachlorophenol.

The amount of stabilizer which must be added to the sand ranges from about 3% by weight of the dry sand for solid grains of intermediate fineness and uniformity to about 7.5% or even 10% for sands possessing large specific surfaces such as those derived from shell or coral.

The solution of the catalysts in the anilene increases the viscosity to values comparable with those of cut-back asphalts. This means that the liquids can be handled at normal temperatures with normal bituminous spreading and mixing machinery. However, because of the need for careful control of the proportions of chemicals added to the soil, considerable difficulty has been encountered in developing staisfactory equipment for applying the anilene-furfural stabilizer in the field.

The most satisfactory results have been obtained using a modified form of the Model 54 Roadmixer of the Wood Manufacturing Company. The unit was towed by a D-8 tractor and travelling at a rate of 17 feet per minute, the machine scooped up beach sand and passed it back through a pug-mill. Here it was thoroughly mixed with the correct amount of chemicals. As the mixed materials emerged from the rear of the pug-mill they were spread and compacted into a strip of any width up to about 12 feet. Eight hours after stabilization, one of these strips easily supported a 38,600 pound load. The cost of the materials used was about 35 cents per sq. ft.

In addition to stabilizing sands, anilene furfural has also proved satisfactory for stabilizing sandy clays and highly calcareous sands. However, for the latter type of materials, higher proportions of stabilizer are required.

It has also been found possible to substitute pitch to some extent for anilene and furfural in the stabilizer. This decreases the brittleness of the stabilized soil and since pitch is both cheaper and more readily available than anilene and furfural, more recent laboratory investigations have been directed to the development of pitch-anilene-furfural stabilizers.

Promising results have also been obtained using a stabilizer formed by combination of pitch with furfural alcohol which is relatively inexpensive and can be made readily available. Further research on this method is in progress.

Although construction difficulties have not yet been completely solved, the use of anilene-furfural or pitch-anilene-furfural combinations represents the most generally satisfactory method for stabilizing beach sands which is available at the present time.

2. Thermite-fusion Method of Beach Sand Stabilization:

In this method of beach sand stabilization, thermite and fluxing agents are used to fuse the sand grains together and form a hard coherent mass. The zones to be stabilized are prepared in four layers; the first (bottom) and third layers consist of thermite and the second and fourth layers of a mixture of sand and flux materials. The thermite is then ignited to fuse the grains together.

Satisfactory results have not been obtained by the thermite method when the ground is not level because the thermite will not burn uphill. Difficulties have also been encountered due to discontinuous and incomplete combustion resulting from excessive moisture, too rapid combustion in the lower thermite layer and eruptions caused by the entrapment of gases formed during the high temperature reaction.

By a suitable construction procedure, a uniform and complete reaction, without discontinuities due to eruptions can be induced in beach sands resulting in a stabilized material which can support traffic. However, erratic results will be obtained in inclement weather or on inclined surfaces. This latter fact makes the method unsuitable for stabilizing the sloping face of a beach.

In the more successful tests, the cost of materials alone was $3.09 per sq. ft., which is approximately six times that of $\frac{1}{4}$ inch steel plate, while the weight of materials required was about twice that of $\frac{1}{4}$ inch steel plate.

In view of the undependable nature of the method, the large amount of additives required and the more promising results obtained in other methods of stabilization, it has been recommended, in a report from the U. S. Naval Civil Engineering Research and Evaluation Laboratory, Port Hueneme, that investigation of the thermite method of beach sand stabilization should be discontinued.

3. Calcium-acrylate Method of Soil Stabilization:

Soil solidification using calcium acrylate is a chemical stabilization process based on the formation of a complex system of soil particles interwoven in and attached to strong flexible polymer chains. When calcium acrylate is mixed with a base-exchangeable soil, the calcium acrylate ions can become attached to the soil particles by replacing the anions of the soil in an exchange reaction; the displaced anions of the soil can react with negatively charged acrylate ions. The next step in the solidification process is to polymerize the acrylate, both that attached to the soil and that which remains dissolved in the water, to form long chain polymers. This is accomplished by the addition of a catalyst (an oxidizing agent) and an activator (a reducing agent). The main polymer chains are attached to the soil through the calcium ions and are also cross-linked through the excess calcium ions. Thus, a complex soil system having considerable strength is produced. The contribution of ion-exchange to the strength of the stabilized soil is minor compared with that of the polymerization. In fact, calcium acrylate has been used successfully to stabilize soils which have no ion exchange capacities.

There are many possible variations in the actual procedure of stabilizing soils with calcium acrylate. The acrylate and the catalyst can be added separately or together; each chemical can be added in powder form, in water solution or in water slurry.

The amount of chemicals required to stabilize a soil depends on the nature of the soil, though percentages varying between 4% and 25% have been successfully used. The redox catalyst system of ammonium persulfate as catalyst and sodium theosulfate as activator (in equal amounts) will satisfactorily effect the polymerization. Percentages of catalyst and of activator from $\frac{1}{2}$% to 20% of the acrylate weight have been used successfully and gel times (elapse time between the addition of the catalyst and the start of polymerization) in the neighborhood of 15 minutes have been obtained in field test section.

Calcium acrylate has been successfully used to solidify a wide variety of soils ranging from beach sands to highly plastic clays. Research is continuously in progress to improve the method, to develop field stabilization procedures and to discover other monomers which will have the same effect as calcium acrylate. At the present time, calcium acrylate is not produced on more than a pilot plant scale.

While the method requires further development, the use of calcium acrylate or a chemical with similar effects, seems to offer the best possibilities for a practical procedure for the stabilization of muds.

Miscellaneous Methods of Constructing Temporary Roadways on Unstable Beach Soils:

At the present stage of development of beach soil stabilization, practical methods have not yet been developed for the satisfactory stabilization of all types and conditions of soil which may be encountered during an amphibious operation. In order that vehicles and equipment may be put ashore across non-trafficable beaches when soil stabilization procedures are not available, methods have been and are being developed for the construction of temporary bridges or roadways stretching from the landing craft to the stable areas on or beyond the backshore.

Temporary roadways may be divided into two main types - pontoon causeways and lightweight mat roadways.

Various types of lightweight roadways have been developed and are being investigated. The Acorn laminated mat was developed during the last war. It weighs about 36 tons per hundred feet of roadway and is laid down by two cranes and a crew of about 35 men.

Other types of roadways in process of development include:

(i) The Swiss Roll. This is essentially a lightweight, planked-over canvas mat designed to carry troops and light vehicles over water and mud.

(ii) The Pneumatic Roadway. This consists of transverse metal channels, in which are collapsible rubber bags. The channels are pin-connected and as they are rolled out the bags are inflated to form pontoons 2 feet in diameter beneath the channels. The roadway weighs about 275 lbs. per lineal foot, can be laid at a rate of about $\frac{1}{4}$ mile per hour and will carry vehicles weighing up to 60 tons.

(iii) The Dory-shaped section roadway. Preliminary plans indicate that the prefabricated steel sections of this roadway, about 40 feet long, will be assembled into 80-foot lengths at a staging area. The cross-section is about 20 feet wide with a 3-foot hollow area below the roadway deck. There will be 3-foot flared sides for added buoyancy and wave protection. If added stability is needed, it is proposed to strap inflatable bags to the outside of the sections.

# MANUAL OF AMPHIBIOUS OCEANOGRAPHY

## SECTION V - BEACH TRAFFICABILITY AND STABILIZATION

### A. BEACHES AND TRAFFICABILITY

## A. BEACHES AND TRAFFICABILITY

One of the most important factors in an amphibious operation is that of getting vehicles from the landing ship or craft across the beach and on to hard terrain. During this process a vehicle must satisfactorly negotiate four different sections of ground:

(i) The submerged section of beach between the landing craft and the shore

(ii) The moist section of the beach (the beach face) which is covered by water at high tide or subject to the waves' uprush

(iii) The dry section of the beach above the limit of uprush at high tide

(iv) The backshore which may be sand dunes, a cliff, a slough, etc.

A common type of beach profile is shown in Figure VA-1. It should be noted that in this figure the vertical scale is greatly exaggerated. A characteristic beach will have the various sections indicated, consisting of a number of bars and troughs, a tidal bench, a step, (all of which comprise the lower beach), an upper beach or beach face, a berm and the backshore. The step is the convex section which coincides with the lowest limit of the uncovered beach just before the new wave breaks; in other words, it is the "limit of backrush".

On the seaward side of the step is the tidal bench which is visible only during the lowest tides. Beyond the tidal bench there may be bars and troughs; a beach may have no bars or there may be many bars of varying heights and widths, depending on the beach and surf conditions.

On the landward side of the step are the beach face, which is partially covered with water at high tide, the berm and the backshore. There may be no berm; it is usually cut back considerably during periods of heavy surf and may on occasions disappear entirely. A vertical scarp usually forms at the top of the beach face when the beach is being cut back. During such period of heavy surf the tidal bench may also disappear.

The beach face is that part of the beach extending from the seaward edge of the berm, if one is present, to the step. It is this section of the beach which is used by vehicles during a landing operation while at the same time it is directly exposed to wave action. The slope of the beach face is determined by the ratio $\frac{h}{w}$ in Figure VA-1. The beach slope classification of the Beach Erosion Board, Corps of Engineers, U. S. Army, is shown in Table VA-1.

Table VA-1

Classification of Beach Slopes

| Description of Slope | Ratio $\frac{h}{w}$ |
|---|---|
| Steep | Greater than 1/15 |
| Moderate | 1/15 to 1/30 |
| Gentle | 1/30 to 1/60 |
| Mild | 1/60 to 1/120 |
| Flat | Less than 1/120 |

Cusps will often occur in the beach face. A cusp is a nearly semi-circular hollow which tends to be at right angles to the sea margin and tapers to a point seaward. These cusps are easily identified on a beach or on aerial photographs, see Figure VA-2. The depth of the hollow of a cusp seems to increase with increasing steepness of the beach face and profiles of a beach may vary considerably depending on whether they are drawn through the crests or hollows of cusps.

It will readily be appreciated that the characteristics of the four main sections of a beach (the submerged section, the moist face, the dry berm and the backshore) will differ considerably with regard to their ability to sustain traffic. Trafficability may be defined as the ability of an area to sustain the motion of wheeled and/or tracked vehicles. While it is possible to evaluate and discuss the trafficability of the different sections of a beach it is evident that all four sections must be trafficable in order to satisfactorily accomplish an amphibious landing operation.

Beach soils vary greatly in character, ranging from cobbles and sands to muds. It is clearly desirable to be able to determine in advance of an amphibious operation whether the beach soil will, or will not be trafficable. When the beach soil is non-trafficable, vehicles may still be enabled to cross the beach by stabilizing sections of the soil or by the construction of temporary roadways or bridges across the beach.

This report describes:

(i) The factors affecting the trafficability of soils.

(ii) Methods for predicting soil trafficability.

(iii) Methods of stabilizing beach soils or constructing temporary roadways across a beach.

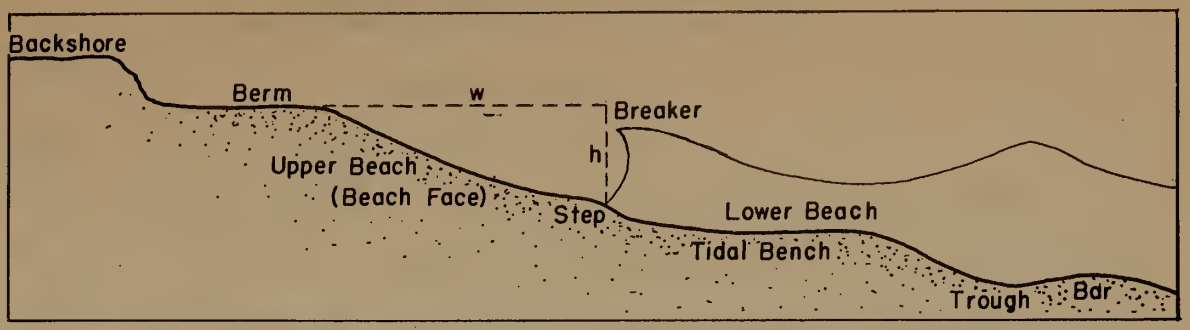
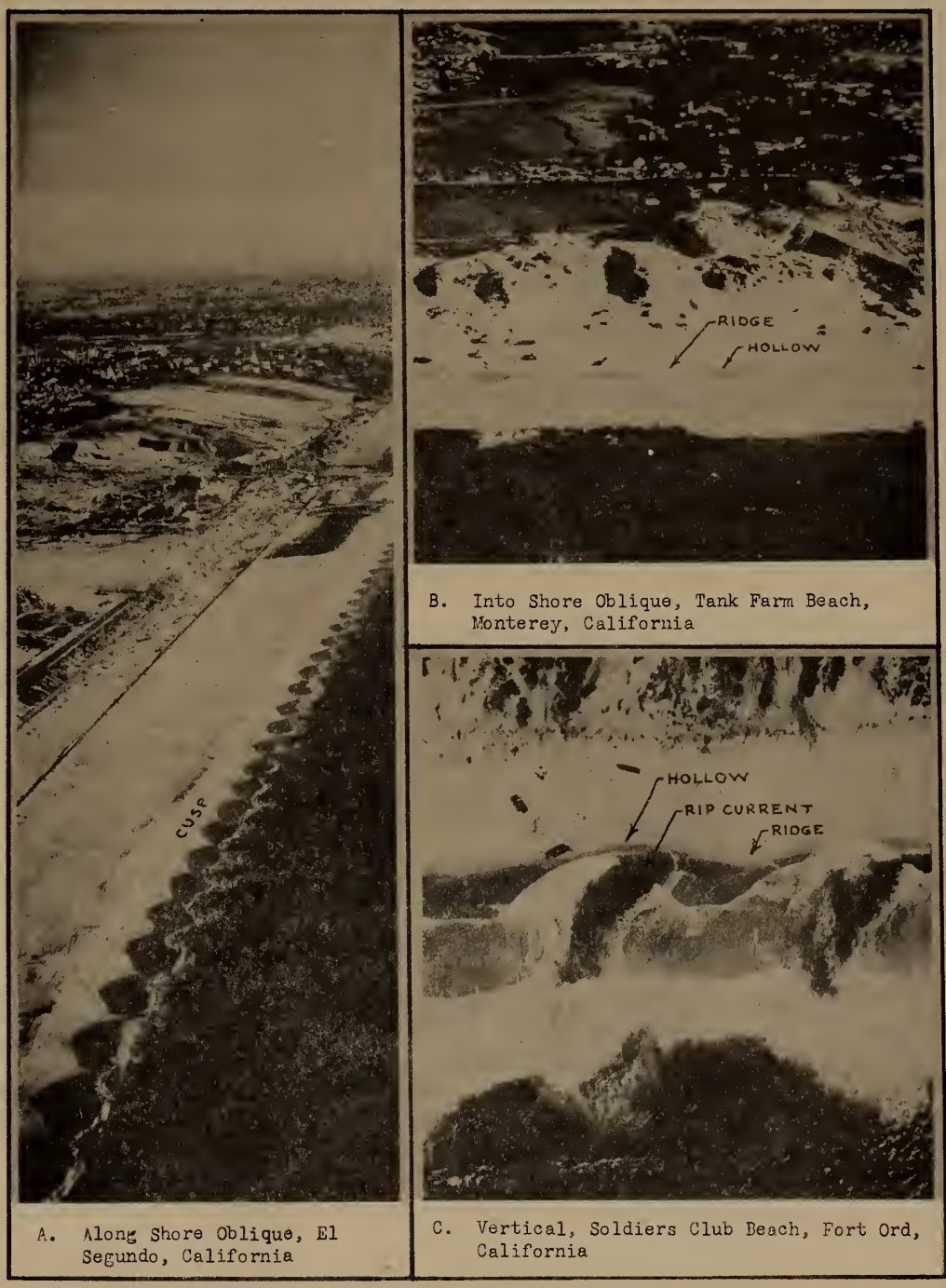



Figure V A-1. COMMON TYPE BEACH PROFILE




A. Along Shore Oblique, El Segundo, California

B. Into Shore Oblique, Tank Farm Beach, Monterey, California

C. Vertical, Soldiers Club Beach, Fort Ord, California

Figure V A-2. CUSPS

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## B. FACTORS AFFECTING TRAFFICABILITY

### 1. General Discussion of Factors Affecting Trafficability

Trafficability has been defined as the ability of an area to sustain the motion of wheeled and/or tracked vehicles. It is evident therefore that the trafficability of an area depends on characteristics of both the area itself and the vehicles considered. It is not proposed to discuss here, in other than a very general way, the vehicle characteristics which affect trafficability, but rather to review the area characteristics which determine whether or not motion of a vehicle is possible. Since trafficability problems on beaches are mainly concerned with areas whose trafficability has not been improved by artificial means, the area characteristics to be considered are mainly those of the natural soil deposit comprising the upper two or three feet of the area.

The trafficability of a soil is dependent on three main factors:

(i) The soil must have sufficient bearing capacity to support the vehicle.

(ii) The soil must have sufficient tractive capacity to develop the resistance, between the soil and the tire or track, required to provide the forward thrust.

(iii) The rolling resistance due to the soil and the vehicle must not be greater than the forward thrust which the vehicle can develop.

Trafficability is also dependent on other factors but only because they cause changes in the three main factors listed above. For example, trafficability may depend on the density of traffic. On some soils the continuous passage of traffic loads over the soil may cause the bearing capacity and traction to decrease and the rolling resistance to increase. If the rolling resistance should become equal to the thrust developed by traction between the treads and the soil a vehicle will be immobilized. Thus, a soil which was initially trafficable may become untrafficable. For this reason, an earlier report of the U. S. Corps of Engineers (VB-12) considered trafficability of soils for two different traffic density conditions:

(i) Limited operation - when vehicle movement was less than 25 repeated trips.

(ii) Unlimited operation - when vehicle movement was greatly in excess of 25 repeated trips.

The ability of a soil to develop adequate bearing capacity depends mainly on the shear strength characteristics of the soil. These in turn depend on the soil type. If a soil has sufficient bearing capacity to support a wheel load on the surface, the deformation under the wheel is mainly due to elastic compression of the soil. However, if the surface bearing capacity is not sufficient to support the load, the wheel may sink into the soil until either the vehicle is supported mainly by flotation or, due to the confining effect of the adjacent soil, the soil below the wheel develops the required bearing capacity. This type of settlement is due to plastic flow resulting from shear failure of the soil. If arrested, the final settlement depends on the shear strength characteristics and the unit weight of the soil.

In muddy areas where the soil is very weak, the first pass of a vehicle causes a rut of considerable depth and successive passes cause additional

deformation and deepening of the ruts. As the depth of the ruts increases it may eventually exceed the ground clearance of the vehicle resulting in a decrease in traction and an increase in rolling resistance. When the rolling resistance is equal to the tractive force the vehicle is immobilized. In such cases the main problem of trafficability is either that of floating the vehicle on the soil or of estimating the number of passes before failure will occur.

The tractive force between the tire or track of a vehicle and the underlying soil is partly developed by the provision of treads (on tires) or grousers (on tracks) which dig into the upper layer and develop thrust due to the passive resistance of the soil to deformation. The maximum passive resistance of a soil is also a function of the shear strength characteristics.

Alternatively, if the track or tire is not able to dig into the soil, the tractive force is developed by friction between the wheel and the soil. Even a very hard material may have a low coefficient of friction, a condition referred to as slipperiness. This condition may also occur when a hard soil is overlain by a thin layer of very soft weak soil which causes an apparent lubricating effect so that little traction is developed.

When a wheel or track sinks into a soil it is necessary for the vehicle to push soil out of the way in order to move forward. This resistance of the soil is an important part of the rolling resistance. If the soil resisting forward movement of the wheel is weak it can be pushed away relatively easily but if it is strong it can withstand considerable pressure. Thus the rolling resistance is a function of the shear strength of the soil.

Other factors affecting rolling resistance are the slope of the surface, the vehicle characteristics and the stickiness of the soil. Under certain conditions some types of soil become sticky and adhere to the tracks and tires of vehicles, accumulating on the track rollers or beneath the fenders or carriage and thus increasing the rolling resistance. This accumulation of sticky soil is termed mud-packing.

Hence, the trafficability of a soil depends on three main factors--bearing capacity, traction capacity and rolling resistance. These factors in turn depend on the following characteristics of the soil:

| | |
|---|---|
| Soil type | Slipperiness |
| Shear strength | Stickiness |
| Unit weight | Friction |

Trafficability also depends on the surface slope, the traffic density and the characteristics of the vehicle. However, not all of these factors need be considered in every case. For example, slipperiness and stickiness have no effect on the trafficability of sands.

Some of the factors affecting trafficability are discussed in further detail in the following pages.

## 2. Effect of Soil Type on Trafficability

Soil type is an important factor in the determination of the trafficability of an area since it affects the strength, slipperiness, and stickiness characteristics of the soil as well as the effects of repeated loads and transient loads

on the soil. Hence, for a given vehicle, the bearing capacity, rolling resistance and traction capacity of the soil will all vary with the soil type.

In a previous report of the Corps of Engineers (VB-12) it was proposed that a modified form of the Casagrande Airfield Soil Classification (VB-3) system be used to classify soils with respect to trafficability. This system is well-suited for trafficability classification purposes. Soils are divided into two main types--coarse grained cohesionless soils such as sands and gravels and fine grained soils such as silts and clays. Sands and gravels are further classified on the basis of grain size distribution and silts and clays on the basis of plasticity characteristics. A complete description of the soil types in this system is shown in Table VB-1.

A modification proposed by the Corps of Engineers in using the Casagrande system for trafficability classification, is to classify fine grained soils on the basis of plasticity index rather than liquid limit. This is suggested because of the apparent correlation of stickiness with plasticity index. However, if some other more satisfactory means of measuring stickiness were developed this modification would be unnecessary.

The coarse grained soils of the system are those comprising sandy beaches, where slipperiness and stickiness are no problem, remolding of the soil does not change its properties and the effect of transient loading on the strength of the soil is relatively small.

In general the fine-grained soils of the system are those encountered on muddy beaches. Here slipperiness and stickiness are soil characteristics which must be included in trafficability considerations, the strength characteristics are quite different from those of sands due to the impervious nature of the soils, the strength of the soil may be reduced by remoulding and under transient loads the strength may be appreciably increased. Furthermore only those soils of very low plasticity, such as inorganic silts, are susceptible to liquefaction in the manner as fine sands.

A knowledge of the soil type is important, therefore, both in assessing the factors affecting the trafficability of the area and in predicting the probable performance of the soil under wheel loads.

3. Stickiness

Under certain conditions some soils are sticky and they will adhere to the treads and tires and accumulate beneath the fenders or carriage, thereby increasing the rolling resistance. Stickiness seems to depend mainly on the soil type and the water content and investigations by the Corps of Engineers (VB-12, 13, and 14) indicated that stickiness is related to the plastic properties of a soil. When the water content of a plastic soil is decreased from a value above the liquid limit, the soil passes through a sticky stage and then into a state where it is not sticky. An attempt was therefore made to correlate the degree of stickiness and the range of water contents at which a soil is sticky with the liquid and plastic limits of the soil.

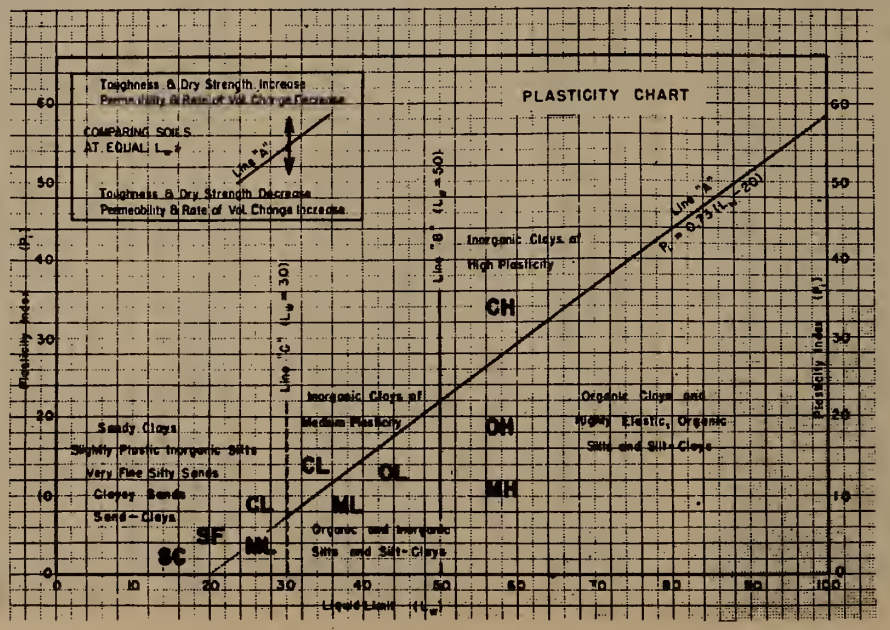
A modified form of Atterberg's stickiness test was used to determine when the soil was in the sticky range. A stainless steel spatula was drawn over the surface of a soil sample at various water contents (Figure VB-1). If no soil adhered the stickiness was noted as "none"; if some soil adhered but areas of the spatula were still exposed the stickiness was noted as "some"; if the

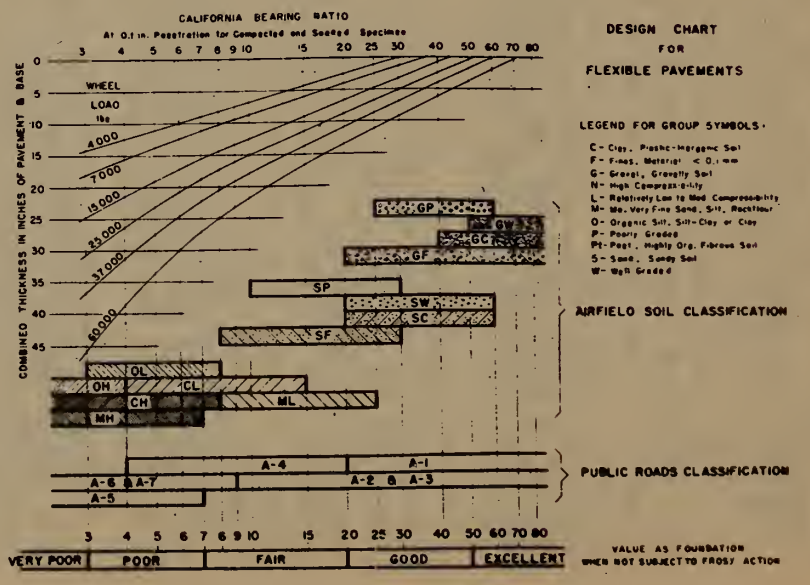
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## TABLE XB-1 SOIL CLASSIFICATION FOR AIRFIELD PROJECTS

| 1 MAJOR DIVISIONS | | 2 SOIL GROUPS & TYPICAL NAMES | 3 SUGGESTED GROUP SYMBOLS | 4 GENERAL IDENTIFICATION: DRY STRENGTH | 4 GENERAL IDENTIFICATION: OTHER PERTINENT EXAMS. | 5 OBSERVATIONS AND TESTS RELATING TO MATERIAL IN PLACE | 6 PRINCIPAL CLASSIFICATION TESTS (ON DISTURBED SAMPLES) | 7 VALUE AS FOUNDATION WHEN NOT SUBJECT TO FROST ACTION | 8 VALUE AS WEARING SURFACE FOR STAGE OR EMERGENCY CONSTRUCTION: WITH DUST PALLIATIVE | 8 VALUE AS WEARING SURFACE FOR STAGE OR EMERGENCY CONSTRUCTION: WITH BITUMINOUS SURFACE TREATMENT | 9 POTENTIAL FROST ACTION | 10 SHRINKAGE, EXPANSION, ELASTICITY | 11 DRAINAGE CHARACTERISTICS | 12 COMPACTION CHARACTERISTICS AND EQUIPMENT | 13* SOLIDS AT OPT COMPACTION LB PER CU FT & VOID RATIO, | 14 CAL BEARING RATIO FOR COMPACTED & SOAKED SPECIMEN | 15 COMPARABLE GROUPS IN PUBLIC ROADS CLASSIFICATION |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| COARSE GRAINED SOILS— | Gravel and Gravelly Soils | Well Graded Gravel & Gravel-Sand Mixtures, Little or No Fines | GW | None | Gradation, Grain Shape | Dry Unit Weight or Void Ratio, Degree of Compaction, Cementation, Durability of Grains, Stratification & Drainage Characteristics, Ground Water Conditions, Traffic Tests, Large Scale Load Tests or California Bearing Tests | Mechanical Analysis | Excellent | Fair to Poor | Excellent | None to Very Slight | Almost None | Excellent | Excellent, Tractor | >125 e<0.35 | >50 | A-3 |
| | | Well Graded Gravel-Sand-Clay Mixtures, Excellent Binder | GC | Medium to High | Gradation, Grain Shape, Binder Exam Wet & Dry | | Mechanical Analysis, Liquid & Plastic Limits on Binder | Excellent | Excellent | Excellent | Medium | Very Slight | Practically Impervious | Excellent, Tamping Roller | >130 e<0.30 | >40 | A-1 |
| | | Poorly Graded Gravel & Gravel-Sand Mixtures, Little or No Fines | GP | None | Gradation, Grain Shape | | Mechanical Analysis | Good to Excellent | Poor | Poor to Fair | None to Very Slight | Almost None | Excellent | Good, Tractor | >115 e<0.45 | 25-60 | A-3 |
| | | Gravel with Fines, Very Silty Gravel, Clayey Gravel, Poorly Graded Gravel-Sand-Clay Mixtures | GF | Very Slight to High | Gradation, Grain Shape, Binder Exam Wet & Dry | | Mechanical Analysis, Liquid & Plastic Limits on Binder if Applicable | Good to Excellent | Poor to Good | Fair to Good | Slight to Medium | Almost None to Slight | Fair to Practically Impervious | Good, Close Control Essential, Rubber Tired Roller, Tractor | >120 e<0.40 | >20 | A-2 |
| | Sands and Sandy Soils | Well Graded Sands & Gravelly Sands, Little or No Fines | SW | None | Gradation, Grain Shape | | Mechanical Analysis | Excellent to Good | Poor | Good | None to Very Slight | Almost None | Excellent | Excellent, Tractor | >120 e<0.40 | 20-60 | A-3 |
| | | Well Graded Sand-Clay Mixtures, Excellent Binder | SC | Medium to High | Gradation, Grain Shape, Binder Exam Wet & Dry | | Mechanical Analysis, Liquid & Plastic Limits on Binder if Applicable | Excellent to Good | Excellent | Excellent | Medium | Very Slight | Practically Impervious | Excellent, Tamping Roller | >125 e<0.35 | 20-60 | A-1 |
| | | Poorly Graded Sands, Little or No Fines | SP | None | Gradation, Grain Shape | | Mechanical Analysis | Fair to Good | Poor | Poor | None to Very Slight | Almost None | Excellent | Good, Tractor | >100 e<0.70 | 10-30 | A-3 |
| | | Sand with Fines, Very Silty Sands, Clayey Sands, Poorly Graded Sand-Clay Mixtures | SF | Very Slight to High | Gradation, Grain Shape, Binder Exam Wet & Dry | | Mechanical Analysis, Liquid & Plastic Limits on Binder if Applicable | Fair to Good | Poor to Good | Poor to Good | Slight to High | Almost None to Medium | Fair to Practically Impervious | Good, Close Control Essential, Rubber Tired Roller | >105 e<0.60 | 8-30 | A-2 |
| FINE GRAINED SOILS Containing Little or No Coarse Grained Material | Fine Grained Soils Having Low to Medium Compressibility | Silts (Inorganic) & Very Fine Sands, Rock Flour, Silty or Clayey Fine Sands with Slight Plasticity | ML | Very Slight to Medium | Examination Wet (Shaking Test & Plasticity) | Dry Unit Weight, Water Content & Void Ratio, Consistency — Undisturbed & Remolded, Stratification, Root Holes, Fissures Etc, Drainage & Ground Water Conditions, Traffic Tests, Large Scale Load Tests, California Bearing Tests or Compression Tests | Mechanical Analysis, Liquid & Plastic Limits if Applicable | Fair to Poor | Poor | | Medium to Very High | Slight to Medium | Fair to Poor | Good to Poor, Close Control Essential, Rubber Tired Roller | >100 e<0.70 | 6-25 | A-4 A-6 A-7 |
| | | Clays (Inorganic) of Low to Medium Plasticity, Sandy Clays, Silty Clays, Lean Clays | CL | Medium to High | Examination in Plastic Range | | Liquid & Plastic Limits | Fair to Poor | Poor | | Medium to High | Medium | Practically Impervious | Fair to Good, Tamping Roller | >100 e<0.70 | 4-15 | A-4 A-6 A-7 |
| | | Organic Silts & Organic Silt-Clays of Low Plasticity | OL | Slight to Medium | Examination in Plastic Range, Odor | | Liquid & Plastic Limits From Natural Condition & After Oven Drying | Poor | Very Poor | | Medium to High | Medium to High | Poor | Fair to Poor, Tamping Roller | >90 e<0.90 | 3-8 | A-4 A-7 |
| | Fine Grained Soils Having High Compressibility | Micaceous or Diatomaceous Fine Sandy & Silty Soils, Elastic Silts | MH | Very Slight to Medium | Examination Wet (Shaking Test & Plasticity) | | Mechanical Analysis Liquid & Plastic Limits if Applicable | Poor | Very Poor | | Medium to Very High | High | Fair to Poor | Poor to Very Poor | <100 e>0.70 | <7 | A-5 |
| | | Clays (Inorganic) of High Plasticity, Fat Clays | CH | High | Examination in Plastic Range | | Liquid & Plastic Limits | Poor to Very Poor | Very Poor | | Medium | High | Practically Impervious | Fair to Poor, Tamping Roller | >90 e<0.90 | <6 | A-6 A-7 |
| | | Organic Clays of Medium to High Plasticity | OH | High | Examination in Plastic Range, Odor | | Liquid & Plastic Limits From Natural Condition & After Oven Drying | Very Poor | Useless | | Medium | High | Practically Impervious | Poor to Very Poor | <100 e>0.70 | <4 | A-7 A-8 |
| Fibrous Organic Soils with Very High Compressibility | | Peat and Other Highly Organic Swamp Soils | Pt | Readily Identified | | Consistency, Texture and Natural Water Content | | Extremely Poor | Useless | | Slight | Very High | Fair to Poor | Compaction Not Practical | | | A-8 |

* THESE WEIGHTS APPLY ONLY TO SOILS HAVING SPECIFIC GRAVITIES FROM 2.65 TO 2.75






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soil completely covered the face of the spatula stickiness was noted as "complete". In this manner the ranges of water contents for "some" and "complete" stickiness were determined.

The test data showed that for soils with a plasticity index greater than 17 there was a water content below which the spatula reading was "none", a range of water contents in which the spatula reading was "some" and a water content above which the spatula reading was "complete". Soils with a plasticity index of 14 or less showed no readings in excess of "some". Four non-plastic soils, a silty sand and two clayey silts showed readings of "none" throughout the entire range of water contents tested.

The water/plasticity ratio or the percent in the plastic range was used as an index of the sticky range. For the soils tested there was little consistency in the value of the percent in the plastic range at which "some" stickiness first occurred. The values range from 0 in the case of a clay from Port Hueneme, for which some stickiness was noted at a water content below the plastic limit, to very high values for soils with low plasticity indexes. There was, however, a reasonable consistency of the percent in the plastic range at which complete stickiness was first noted. The values ranged from 26 to 71 percent. For the silty and sandy clays the values ranged between 44 and 71 percent, while for the clays the values ranged between 26 and 40 percent.

Some of the soils on which laboratory tests were performed were also tested for trafficability in the field. It was found that the spatula test was not too successful in predicting whether or not the soil would stick to a vehicle. When the spatula reading was "none" or "some" no stickiness of soil to vehicle occurred; when the spatula reading was "incomplete", in some cases stickiness of the soil to a vehicle was reported and in other cases no stickiness was observed. This indicates that the soils with low plasticity index values which did not show "complete" stickiness in the spatula tests, will cause less trouble from stickiness than the soils with higher plasticity index values.

Another disadvantage of the spatula test is that it does not indicate the degree of stickiness. The only means for determining the relative stickiness of soils is the approximate correlation observed in the tests described above and in other fields, that the higher the plasticity index, the stickier the soil.

Results are available (VB-13) for one field test area to indicate the percent in the plastic range at which stickiness of the soil to a vehicle was first observed. At this site the soil was classified CH and stickiness was only reported in trafficability tests when the water content was greater than 34% in the plastic range. This result indicates that the value of 50% determined in earlier correlations (VB-12) as the approximate point in the plastic range at which stickiness begins to develop is probably too low.

Field test results for two sites where the soils were classified ML and CH are also reported. Trafficability tests were conducted using a D-7 tractor, a $2\frac{1}{2}$ ton truck, a Weasel M29C and a L.V.T.-4 with water contents for each soil ranging between the liquid and plastic limits. The results showed the Weasel to be the only vehicle immoblized by stickiness, and this occurred in both soils. Since the ML soil was not a highly sticky soil this immobilization of the Weasel was considered due to some characteristic of the vehicle rather than to stickiness of the soil. However, this should not be construed to mean that stickiness is not an important factor in trafficability. Cases have been known where vehicles

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were immoblized from stickiness where the mud was very shallow.

[illegible] the stickiness characteristics of soils, with the possible development of a new test for measuring stickiness, is required before the effects of this factor can be satisfactorily included in an analysis of soil trafficability.

4. Slipperiness

When a thin layer of weak plastic soil overlies a hard soil, certain types of tires and tracks are not able to dig into the hard soil and the weak soil has an apparent lubricating effect so that no traction is developed. This condition is called slipperiness. An excellent example of the way in which slipperiness can immoblize a vehicle is reported by the Corps of Engineers (VB-13). A trafficability test using a $2\frac{1}{2}$ ton truck was conducted on a soil in the ML group. Although the soil was sufficiently strong to support the vehicle, accidental spillage of water on the surface made it so slippery that rubber tires without chains could not develop the necessary traction.

An attempt has been made by the Corps of Engineers to correlate slipperiness with the plasticity limits of soils. The following results are reported (VB-12) from trafficability tests using $2\frac{1}{2}$, 4 and 6 ton trucks, D-7 and D-8 tractors and 6 ton Athey crawler wagons on ML and CL type soils. Correlation of slipperiness with trafficability was not very definite. For rubber tires the data indicate that practically any moisture content within or above the plastic range is too slippery for the vehicles to negotiate. This is particularly true if the soil, although firm and at a relatively low moisture content, is covered with a little water or fluid mud. In the traffic tests practically all trucks without chains were immoblized on the first pass in the sections where water was standing. Tractors and trucks with chains were able to cut down to firm material and obtain sufficient traction to negotiate. For tires with traction devices and tracked vehicles the slippery range is just beyond the sticky range.

5. Rolling Resistance

When a relatively rigid wheel rolls on a non-elastic surface the surface is deformed as shown in Figure VB-2. The material is continuously compressed and depressed ahead of and below the wheel resulting in greater rolling resistance than would develop on an elastic surface.

Tests (VB-7) have shown that a rigid wheel causes a permanent soil displacement parallel as well as perpendicular to the direction of travel, that is, the soil surrounding the track left by a wheel has been moved ahead as well as at right angles to the direction of travel.

The total rolling resistance may be divided into two main parts; internal rolling resistance in the vehicle itself and external rolling resistance due to the soil.

External rolling resistance is due to:

(i) Resistance to displacement of soil forward, laterally and downward.

(ii) Friction or adhesion between the wheels and the soil.

(a) at the face of the rim
(b) at the edges of the rim



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(iii) Friction or adhesion between the soil and the undercarriage of the vehicle if the depth of sinkage exceeds the ground clearance.

(iv) Adhesion of soil to the wheels of the vehicle resulting in mud-packing.

Internal rolling resistance is due to mechanical features of the vehicle, mainly friction in the axle bearings.

With the exception of adhesion which remains constant for a given soil condition, an increase in load increases the magnitude of the other factors affecting rolling resistance. Thus for a given wheel and soil condition, increased load means increased rolling resistance. It is also indicated that for a given wheel width and load and given soil conditions, an increase in wheel diameter causes a reduction in rolling resistance.

Slippage is said to occur when there is relative motion between the wheel and the soil at the points of contact. In such cases the vehicle does not move forward through a distance equal to the circumference of the wheel for each complete revolution of the wheel. In general, any wheel or soil change which tends to increase rolling resistance also tends to increase slippage. However, where the surface layer has a lower supporting capacity than the underlying soil this may not be true. Under such circumstances an increase in load may cause the firmer stratum to become more effective and thus tend to decrease slippage.

## 6. Strength Characteristics of Soils

### a. Cohesionless Sands

It was in the eighteenth century that Coulomb established the relationship between the shear strength of sand and the normal pressure. If sand is placed in a box as shown in Figure VB-3, subjected to a vertical stress p and then the upper part of the box is pulled horizontally to shear the sand along the plane QQ, the relationship between the shear strength s and the normal stress p on the failure plane is obtained. By repeating the test using different values of p, a general relationship between s and p may be determined. Coulomb showed that for a given sand this relationship is expressed by the equation

$$s = p \tan \phi$$

If this relationship, which is known as Coulomb's law is plotted as shown in Figure VB-4, it defines a straight line passing through the origin and inclined at an angle $\phi$ to the p-axis. The angle $\phi$ is called the angle of internal friction. The greater the normal pressure on the failure plane the greater the frictional resistance which is developed between the sand grains and the greater is the shear strength.

An angle of internal friction is not a constant for all sands but seems to depend mainly on the degree of uniformity of the sand, the angularity of the grains and the degree of compaction or the void ratio. The value of $\phi$ decreases with increasing uniformity of the sand grains and increases with increasing angularity of the grains. For a given sand, in which the grain shape and distribution are fixed, the value of $\phi$ still varies with the void ratio. The lower the void ratio (the more compact the sand) the higher is the angle of internal friction.



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A typical average value of the angle of internal friction of a sand deposit is about 34° but the variation in $\phi$ values of a sand in very loose and very dense conditions may be as much as 10°. It is clear that the higher the angle of internal friction of a sand the greater is the shear strength.

The strength of a sand also varies slightly with the rate of load application or the time of loading. For times of loading greater than about 5 minutes there is no appreciable effect but a change in time of loading from about 5 minutes to .02 seconds causes an increase in strength of about 20 percent.

b. Saturated Clays

The shear strength characteristics of clays constitutes one of the most difficult and least understood subject of soil mechanics. As with sands, the strength of a clay depends mainly on the effective intergranular pressure, which in a clay may vary with time due to the slow rate of consolidation. Thus, under a given stress condition, a saturated clay may have different strengths, corresponding to different effective stresses, depending on the degree of consolidation.

Since the water content of a saturated clay is also dependent on the effective pressure, the shear strength can be related to the water content. For a given rate of loading, all available information indicates that the strength of a given clay depends only on the water content.

For the higher rates of loading the strength of a clay, even at constant water content, varies with the rate of load application. However, when the time of loading exceeds about 5 minutes, this effect is of negligible proportions.

The strength of a clay also depends on certain inherent characteristics of the clay itself. Fundamentally these may be reduced to structural, frictional and cohesive characteristics. For a given material in its natural state, these properties cannot be controlled by external agencies.

In many natural clay deposits the clay has a characteristic structure. Such clays exhibit a loss of strength after failure and when non-uniform stress conditions exist in such a material, progressive failure is an important consideration in determining the over-all strength.

Destruction of the natural structure of a clay by remoulding results in a marked change in the strength and compression characteristics. For example, Boston blue clay, having a moderately well-developed structure, is usually medium stiff and brittle in the natural state but becomes soft and sticky when remoulded at the same water content and the strength may be reduced to only about one tenth of its original value. When a clay possesses a natural structure, it is important to preserve this structure in order to derive maximum benefit from the strength characteristics.

Not all clays possess a natural structure and it is the clays in this group which are likely to present trafficability problems at beaches. These clays will be plastic in character, showing no loss of strength after failure, and will not be affected by progressive failure. Furthermore, the strength will not be affected appreciably by remoulding.

The shearing resistance of a clay is due partly to friction effects, as in sands, and also to cohesion. The cohesion may be what is termed apparent cohesion, when it is actually due to frictional resistance developed by the menisci formed



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in the capillaries at the surface of the soil, or it may be true cohesion which is evidenced even when the menisci are destroyed by submergence of the soil in water. However, it should be emphasized that the cohesion is not a constant for a given soil. The longer a clay is immersed in water, in an unconfined state, the lower becomes the cohesion and after years or decades it may be reduced to zero.

A saturated clay at constant water content has the same shear strength whatever the confining pressure on the clay. Hence, the material behaves as if there were no frictional resistance and the shear strength may be expressed by the formula

$$s = c$$

as if all the strength were due to cohesion (c). Since the particular value of the cohesion applies only at the particular water content of the clay, it is termed the effective cohesion corresponding to these particular loading conditions.

When a saturated clay is tested under conditions such that different normal stresses can produce corresponding changes in the water content, the shear strength normal stress relationship may be approximated by a straight line (Figure VB-5) of the form

$$s = c + p \tan \phi$$

Again since the values of c and $\phi$ are not constants and would apply only to the particular method of test they are termed the effective cohesion and the effective angle of friction corresponding to the particular loading conditions. The above relationship is known as Coulomb's law for the shear strength of a soil. It is important to recognize, however, that the terms c and $\phi$ in this relationship may vary considerably depending on the loading conditions. It is impossible therefore, to convey any indication of the shear strength properties of a clay by reference to the c and the $\phi$ values without a qualifying statement regarding the procedure used to obtain these values. No single set of c and $\phi$ values are in themselves characteristic of the strength properties. Rather c and $\phi$ should be regarded as empirical coefficients which vary with the conditions but which, when carefully selected for any given problem, enable the shear strength to be expressed in a simple and convenient form.

For the saturated clays encountered in trafficability problems on beaches the effective angle of friction will invariably be equal to zero and the shear strength will be expressed by the simplified formula, $s = c$.

c. Partially Saturated Clays and Sand-clay Mixtures

The strength of partially saturated clays and sand-clay mixtures is again due partially to frictional and partly to cohesive characteristics. The relationship between shear strength and normal pressure may be closely approximated by the formula:

$$s = c + p \tan \phi$$

where c and $\phi$ are the effective cohesion and the effective angle of friction respectively which pertain to the particular problem (Figure VB-5). The

appropriate values of c and $\phi$ are determined by tests in which the procedure is selected to reproduce as closely as possible the condition in the problem being investigated.

## 7. Bearing Capacity of Soils

The stresses in a soil caused by moving wheel loads have been found to be essentially the same as those caused by a strip load. Many analyses have been proposed for determining the bearing capacity of soils under these loading conditions. Some of these analyses apply only to cohesive soils in which the shear strength is independent of the normal pressure, some apply only to cohesionless soils while others are derived for soils possessing both frictional and cohesive characteristics. The resulting formulas, for clays, have been shown to agree closely with the actual bearing capacities determined in tests but recent test data, for footings on soils for which the strength is determined mainly by friction, indicated that even the best formulas for the bearing capacity of these materials cannot be considered too reliable.

A summary of some of the more satisfactory analyses, by Krey, Prandtl, and Terzaghi, for the ultimate bearing capacity of strip footings is presented below. The analysis by Krey applies only to cohesive soils in which the shear strength is independent of the normal pressure but those by Prandtl and Terzaghi can be applied to any type of soil.

### a. Krey Analysis for Surface Strip Load (VB-5)

In this method of analysis it is assumed that

(i) The soil has a constant shear strength, c.

(ii) There is no loss of shear strength in the soil after failure.

(iii) Failure takes place by rotation about the center of a cylindrical sliding surface in the soil.

A centre of rotation is selected and the cylindrical sliding surface passing through one end of the footing is drawn as shown in Figure VB-6. The unit load on the footing which will cause failure along this sliding surface is then determined by equating the moment of the foot pressure about the centre of rotation to the moment of the shearing resistance along the sliding surface. For example, if the centre of rotation is selected to be at one end of the footing, as shown in the Figure VB-7, the unit load, q, required to cause failure is determined from the moment equation:

$$(q \times b) \times \frac{b}{2} = (\pi \times b \times c) \times b$$

$$\text{i.e. } q = 2\pi c$$

However, this surface is not that on which failure will occur. By repeated trials of different possible sliding surfaces it may be shown that on the cylindrical surface shown in Figure VB-6, the unit load required to cause failure by rotation is only 5.5 c, and the unit load required to cause failure on any other possible sliding surface is greater than 5.5 c. Hence, failure will occur on the surface shown in Figure VB-6 and the corresponding bearing capacity is

$$q_u = 5.5c$$

b. Prandtl Analysis (VB-8)

In this analysis it is considered that failure develops simultaneously on both sides of the loaded area, the mode of failure being as indicated in Figure VB-8. The main assumptions in the derivation are as follows:

(i) The soil is weightless

(ii) The shear strength vs. normal pressure relationship of the soil is of the form $s = c + p \tan \phi$.

(iii) Wedges ABC, ADE, and BFG behave as rigid masses and move bodily without deformation, sectors ACD and BCF deform plastically while the remainder of the soil is essentially unaffected. The wedge ABC is pushed downwards and the wedges ADE and BFG are pushed upwards obliquely by the pressures transferred through the plastic sectors.

(iv) In the plastic sectors, the stresses are constant along any radius sector such as BH but vary with the angle $\theta$.

From a consideration of the equilibrium of the plastic sectors and the known boundary conditions that along AC and BC the major principal stress is $q_u$ and along AD and BF the minor principal stress is zero, the following equation for the ultimate bearing capacity of the soil is obtained.

$$q_u = \frac{c}{\tan \phi}\left[e^{\pi \tan \phi} \tan^2 (45 + \frac{\phi}{2}) - 1\right]$$

It will be seen that if $c = 0$, $q_u$ becomes equal to zero. This would mean that a cohesionless soil, such as dry sand, has no bearing capacity. This is not the case and the assumption mainly responsible for the discrepancy in the soil is weightless. The following modifications of the Prandtl formula, to correct for this error, have therefore been proposed by Terzaghi and Taylor, the former being probably the more accurate.

(1) Modification by Terzaghi (VB-6). In order to account for the weight of the soil the term c in the original formula should be increased by $c'$ where

$$c' = n \gamma \tan \phi$$

$$n = \frac{\text{Area of wedges and sectors}}{\text{Length EG}}$$

and $\gamma$ = unit weight of soil

The formula then becomes:

$$q_u = \frac{c + c'}{\tan \phi}\left[e^{\pi \tan \phi} \tan^2 (45 + \frac{\phi}{2}) - 1\right]$$

(2) Modification by Taylor (VB-10). To allow for the weight of the soil the term $\frac{c}{\tan \phi}$ in the original formula should be increased by

$$\frac{b\gamma}{2} \cot (45 - \frac{\phi}{2}) \text{ where}$$

b = width of loaded area

and $\gamma$ = unit weight of soil

The formula then becomes:

$$q_u = \left[c \cot \phi + \tfrac{1}{2} b \gamma \tan \left(45 + \frac{\phi}{2}\right)\right]\left[e^{\pi \tan \phi} \tan^2 \left(45 + \frac{\phi}{2}\right) - 1\right]$$

The above formulas are derived for loaded area on the surface of a soil. If the loaded area is at some depth below the adjacent ground surface there is a further increase in bearing capacity. This increase is small for cohesive soils of constant shear strength but is considerable for cohesionless soils. It has been shown that the increase in bearing capacity, above that given by the preceeding formulas, when the footing is at a depth d below the adjacent ground surfaces is equal to

$$\gamma d \left[e^{\pi \tan \phi} \tan^2 \left(45 + \frac{\phi}{2}\right)\right]$$

for $\phi = o$, this expression reduces to $\gamma d$ but for $\phi = 35^o$ it becomes $32 \gamma d$. Hence, using the above Terzaghi expression, the modified Prandtl formula may be expressed in the general form:

$$q_u = \left[\frac{c + c'}{\tan \phi} + \gamma d\right]\left[e^{\pi \tan \phi} \tan^2 \left(45 + \frac{\phi}{2}\right)\right] - \frac{c + c'}{\tan \phi}$$

Using the Taylor modification, the general form of the Prandtl formula becomes:

$$q_u = \left[c \cot \phi + \tfrac{1}{2} b \gamma \tan\left(45 + \frac{\phi}{2}\right) + \gamma d\right]\left[e^{\pi \tan \phi} \tan^2 \left(45 + \frac{\phi}{2}\right)\right] - c \cot \phi - \tfrac{1}{2} b \gamma \tan\left(45 + \frac{\phi}{2}\right)$$

It is of interest to consider the values of these expressions for special cases. For example, for a soil in which the shear strength is independent of the normal pressure, (i.e. s = c and $\phi = o$), the latter equation reduces to:

$$q_u = 5.14c + \gamma d$$

and thus for a surface load (d = o) the bearing capacity is 5.14c.

Again, for a cohesionless soil for which $\phi = 35$, the bearing capacity computed from the Taylor modification of the Prandtl formula is:

$$q_u = 30 b \gamma + 32 \gamma d$$

It will be seen that on a plastic clay soil the depth d of the loaded area below the ground surface increases the bearing capacity by only $\gamma d$ compared with a corresponding increase of about $32 \gamma d$ on a cohesionless soil.

c. Terzaghi Analysis

The analysis of the ultimate bearing capacity of a strip footing by Terzaghi (VB-11) is also applicable to all types of soil. Two failure conditions are recognized. Failure by general shear is considered to occur when settlement increases relatively abruptly over a small range of stress and then increases progressively without further increase in stress, the stress-settlement curve having the form I in Figure VB-9. When failure is due to local shear, settlements are relatively large and there is no definite point of failure; the

stress-settlement curve has the general form of curve II in Figure VB-9 and failure is arbitrarily considered to have occurred when the curve passes into a steep and fairly straight tangent. Failure due to general shear occurs mainly in fairly dense and stiff soils while loose or soft soils are more apt to fail by local shear.

The following formulas for the bearing capacity of a strip footing are obtained:

Failure due to general shear: $q_u = cN_c + \frac{1}{2}\gamma bN_\gamma + \gamma dN_q$

Failure due to local shear: $q_u = 2/3cN'_c + \frac{1}{2}\gamma bN'_\gamma + \gamma dN'_q$

In these formulas $N_c$, $N_\gamma$, $N_q$, $N'_c$, $N'_\gamma$, and $N'_q$ are dimensionless bearing capacity factors depending only on the value of $\phi$; their values may be obtained from charts. The terms expressing the effect of the depth of the footing below the adjacent ground surfaces are only valid when d is less than b.

For a cohesive soil having a constant shear strength $s = c$, the above formulas reduce to:

Failure by general shear: $q_u = 5.7c + d\gamma$

Failure by local shear: $q_u = 3.8c + d\gamma$

For a cohesionless soil having a $\phi$ value of 35° the bearing capacities computed from the Terzaghi formulas are:

Failure by general shear: $21b\gamma + 40d\gamma$

Failure by local shear: $10b\gamma + 13d\gamma$

It is recognized that the results obtained by this method of analysis are both approximate and conservative.

d. Comparison of Results by Different Analysis

The bearing capacity values computed by the methods previously described are summarized below for the special cases of cohesive soils for which the shear strength is independent of the normal pressure and cohesionless soils.

Table VB-2

Cohesive Soils, Shear Strength $s = c$.

| Method of Analysis | | Bearing Capacity of Strip Footing | |
|---|---|---|---|
| | | Surface | Depth d below adjacent soil surfaces |
| Krey | | $q_u = 5.5c$ | |
| Prandtl | | $q_u = 5.14c$ | $q_u = 5.14c + \gamma d$ |
| Terzaghi | General shear | $q_u = 5.7c$ | $q_u = 5.7c + \gamma d$ |
| | Local shear | $q_u = 3.8c$ | $q_u = 3.8c + \gamma d$ |

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Table VB-3

Cohesionless Soils, $\phi = 35^{\circ}$

| Method of Analysis | | Bearing capacity of strip footing, width b, and depth d below adjacent ground surfaces |
|---|---|---|
| Prandtl | | $q_u = 30\,\gamma' b + 32\,\gamma' d$ |
| Terzaghi | General shear | $q_u = 21\,\gamma' b + 40\,\gamma' d$ |
| | Local shear | $q_u = 10\,\gamma' b + 13\,\gamma' d$ |

It will be seen from Table VB-2 that there is good general agreement between the various formulas for the bearing capacity of a soil having a shear strength independent of the normal pressure. In general, the bearing capacity of such a soil for a surface strip load is about 5c and the increase in bearing capacity, $\gamma' d$, when the loaded area is below the adjacent ground surface is relatively small. These results have been shown to be in good agreement with laboratory test and field measurements of bearing capacity.

The different analyses show considerable variation in the bearing capacity values of cohesionless soils (Table VB-3). For surface loading the bearing capacity computed from the Prandtl formula is about 50% greater than the value given by the Terzaghi formula for general shear failure. Recent tests on cohesionless soils have indicated that bearing capacity values computed from any of these formulas are appreciably lower than the test values, thus confirming the previously known fact that the formulas are very conservative.

It is worthy of note that the bearing capacity increases rapidly with increasing values of $\phi$. Hence, a small increase in compaction of a soil, resulting in an increase in the angle of friction, will cause a considerable increase in bearing capacity. Again, the bearing capacity is considerably increased if the loaded area is below the adjacent ground surface; for a cohesionless soil this increase is of the order of $35\,\gamma' d$ compared with only $\gamma' d$ for a soil for which the effective angle of friction is zero.

The bearing capacity of a cohesionless soil is considerably reduced if the ground water table is at or near the ground surface. In such a case the effective unit weight $\gamma'$ of the soil is the buoyant unit weight of the soil grains. This value may be less than one-half the unit weight of the soil resulting, since bearing capacity is directly proportional to unit weight, in a corresponding reduction in bearing capacity. Hence, the bearing capacity of a submerged sand is only about one-half of that of the same sand above the water table.

The bearing capacity of a cohesionless soil is also proportional to the width of the loaded area. The same sand will have twice the bearing capacity under an 8" wide footing on the surface as it will under a 4" wide footing. This explains the extremely beneficial effects of reducing the tire pressure of vehicles travelling on sands. Very often, with a tire pressure of 20 psi a vehicle will become stuck in the sand yet if the tire pressure is reduced to 10 psi the vehicle can operate. In such a case the vehicle is immoblized due

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to insufficient bearing capacity of the soil; a reduction in tire pressure has two effects:

(i) the required bearing capacity, which is equal to the tire pressure, is reduced;

(ii) the contact area of the tire with the ground is increased resulting in a greater width of the loaded area and a corresponding increase in bearing capacity.

The combined value of these effects may result in sufficient bearing capacity of the soil to permit vehicle operation.

8. Effect of Water Content of Sand on Trafficability

In a dry state the bearing capacity of sand depends mainly on the angle of friction of the sand and the size of the loaded area. In the moist state the bearing capacity is increased due to the apparent cohesion of the sand mass. This apparent cohesion is due to the surface tension in the water films which form at the points of contact between sand grains, causing an increase in intergranular pressure and a corresponding increase in strength.

If a capillary tube is placed with its lower end in water, the water will rise in the tube to a height, $h_c$, which is called the height of capillary rise (Figure VB-10). The rise of the water in the tube is attributed to the phenomenon of surface tension at the water surface. The existence of surface tension can be demonstrated by numerous simple experiments. For example, a greasy sewing needle can be made to float on the water. The water surface seems to behave as a thin membrane and curves down under the needle until the upward component of the surface tension is large enough to support the weight of the needle (Figure VB-11). Actually, the surface behaves as a special kind of membrane in which the tension is constant regardless of the amount of deformation. This tension force, $T_s$, can be measured and for water has a value of 0.076 grams/cm of length.

Because of the affinity between water and glass, water will rise in a capillary tube until the vertical component of the surface tension force in the water meniscus is just balanced by the weight of the water column. For a moist glass surface the angle of contact between the meniscus and the glass is zero. Thus in a glass capillary tube there is an upward force of magnitude $T_s$ per unit length acting all around the perimeter of the meniscus and supporting the column of water below. The total upward force is therefore $2\pi r_t \times T_s$. If $h_c$ is the height of capillary rise, the weight of the column of water is $\pi r_t^2 h_c \times \gamma_w$. Hence (Figure VB-12)

$$2\pi r_t T_s = \pi r_t^2 h_c \gamma_w$$

and

$$h_c = \frac{2T_s}{r_t \gamma_w}$$

$$= \frac{4T_s}{d_t \gamma_w}$$

In the metric system, $T_s = 0.076$ gm/cm and $\gamma_w = 1$ gm/cc

$$\therefore h_c = \frac{4 \times 0.076}{d_t \times 1} = \frac{0.3}{d_t} \text{ cm.}$$

In effect, the column of water of height $h_c$ is hanging on the meniscus and this in turn is hanging on the walls of the glass tube. The water is therefore in tension and the walls of the tube are in compression. The concept of water being in tension is not always easy to appreciate but actually water has an extremely high tensile strength, greater than that of steel.

The tensile stress, u, in the water just below the meniscus is equal to the weight of the column per unit area. Hence

$$u = \frac{\pi r_t^2 h_c \gamma_w}{\pi r_t^2} = h_c \gamma_w = \frac{2T_s}{r_t}$$

Thus the smaller the diameter of the capillary, the greater is the tensile stress in the water. A corresponding compressive stress is developed in the walls of the capillary tube.

An effect similar to that in capillary tubes occurs in soils. Here the continuous voids between the grains form the capillaries and the grains themselves constitute the walls of the tube. Thus in soils there is a capillary fringe just above the ground water table; in this zone the ground water rises above the water table to a height equal to the height of capillary rise corresponding to the effective diameter of the pores in the soil. The smaller the grains and the more well graded the soil, the smaller are the pores and the greater is the height of capillary rise. In sands the rise may be only one or two inches, in silts it may be several feet and in clays it may be tens of feet. In the capillary fringe the water is in tension; there is a corresponding compressive stress between the grains and in consequence, an increased frictional resistance to deformation. The smaller the diameter of the soil grains, the smaller is the diameter of the capillaries, the greater is the tensile stress in the water and the greater is the resulting intergranular pressure between the soil grains.

The effects of surface tension in menisci can make an important contribution to the strength of soils. Wherever an air-water interface exists there is a surface in which surface tension effects occur. When menisci are formed the surface tension in the menisci results in a compressive force between the grains and an increase in frictional resistance of the soil. For example, in a moist soil, menisci are formed at the points of contact between grains as shown in Figure VB-13. The surface tension in the menisci pulls the grains together developing an increased pressure between the grains and increased frictional strength.

It is the frictional strength due to the menisci which cause the phenomenon of bulking in moist sand. A moist sand, when handled, will assume a larger volume than the loosest volume in either the dry or completely saturated states. When a sand is deformed the grains move one over another. In a moist sand the grains are not so free to slide as they are in a dry sand because of the frictional resistance developed by the menisci and as a result they do not settle to form so dense a mass. However, if a moist sand in a bulked condition is saturated the menisci are destroyed, the surface tension no longer pulls the grains together and the material slumps to a more compact condition.

One of the more important effects of saturating a soil is the destruction of the menisci between the grains and the resulting loss of strength. A moist sand will possess some shear strength due to the menisci even though no external pressure is applied to the sand. The strength resulting from this apparently

invisible source of intergranular pressure is called apparent cohesion. The apparent cohesion is completely lost if the sand is dried or submerged.

On a beach it is the apparent cohesion of the moist sand which makes the moist section more trafficable than others. Even close to the water's edge, menisci will tend to form a few seconds after the water has receded and apparent cohesion will rapidly develop. In consequence, provided liquefaction does not occur, the moist section of the beach will have a higher bearing capacity than either the dry or submerged sections.

This was found in the trafficability studies carried out by the University of California (VB-15). In general, it was found that the higher the water content of a beach, the better was the trafficability and the best trafficability was found to occur on the wet section of a beach exposed during low tide. It was also found that wheeled vehicles could operate on the normally dry sand of the berm much more easily after a heavy rainfall.

The greater bearing capacity of wet sand on the beach face as compared with relatively dry sand on the berm is indicated by the depth of the tracks left by a DUKW in tests at Clatsop Spit Beach, Fort Stevens, Oregon (Figure VB-14). The better trafficability of moist sand is also illustrated by the data in Figure VB-15. It will be seen that the maximum DUKW velocity on moist sand is between 2 and 3 times greater than the maximum velocity on dry sand.

It should be noted that the coarser the grains of a beach sand, the larger will be the curvature of the menisci and the smaller will be the apparent cohesion. Thus it would be expected that moisture will be a much more important factor in increasing the trafficability of beaches where the sand grains are relatively fine than at beaches where the particles are predominantly of coarse sand or gravel sizes.

9. Liquefaction of Sand

Deformation of a loose sand causes a decrease in volume of the mass. In a loose deposit, although the sand may be able to sustain quite a high load, the grains are not in their most stable conditions and deformation of the mass will cause them to fall into more stable positions. As a result the grains become more closely packed together, the volume of voids is reduced and therefore the volume of the sand mass is reduced.

The process of densifying a soil by elimination of air from the voids is termed compaction of the soil. The best method of compacting a sand is by vibration. The shaking of the grains causes them to fall into positions which are increasing more stable until finally no further decrease in void ratio can be attained. Static pressure applied to a sand does not produce this effect.

The comparative effects of static pressure and vibration in compacting sands may be illustrated by a simple demonstration. If a graduated glass cylinder is filled with 1000 cc. of loose sand and a piston placed on top of the sand, it will be found that no matter how hard one pushes on the piston, no measurable change in volume of the sand can be produced. However, if the side of the cylinder is gently tapped, the volume of the sand can easily be reduced by about 5%.

In the construction of embankments, pavements and foundations the compaction of a sand in order to increase its stability is extremely important. In such

cases where the sand is dry or only partially saturated, the effects of vibrations are beneficial. However, the effects of vibrations on a saturated deposit of loose sand may cause the mass to become completely unstable.

In a saturated loose condition a sand is able to withstand a certain amount of static pressure. The grains of the sand have some structural arrangement which develops strength by friction between the grains and is stable provided the structure is not deformed. If such a deposit is vibrated, however, the grains immediately tend to fall into more stable positions, displacing water from the voids and the structure collapses suddenly. However, since time is required for water to drain from the mass the grains are unable to come in contact due to the excess water in the voids and instantaneously any load on the soil must be carried by the water. As a result the entire mass behaves as a fluid. Any object on the surface of the mass immediately sinks or if the mass constitutes an embankment a "flow slide" occurs. In this way a large bank of soil may suddenly liquefy and spread itself over a large area. Ultimately the movement may be arrested by the drainage of excess water, as occurs in model experiments in the laboratory, or on a large scale it may continue until an entire embankment has practically disappeared.

From the point of view of trafficability, the important effect of liquefaction is the loss of supporting capacity of the soil. On a beach, the saturated sand near the water's edge may be caused to liquefy locally due to the vibrations produced by vehicular traffic; as a result the vehicle will sink and become stuck. The degree of liquefaction resulting from such vibrations will, of course, depend on the initial degree of compaction and saturation of the beach material. For the purpose of estimating the possible range of liquefaction, the obtaining of reliable data on the density of beach deposits would be extremely helpful.

Experience has shown that in general the trafficability of the saturated sand of a beach in the uprush during a low tide is almost as good as that of the moist sand on the beach face. This may well be due to a relatively high degree of compaction of beach sands in that section of the beach submerged at high tide. Saturation of a sand is itself often used as a method of compaction and on a beach the beneficial effects of saturation are combined with the continual vibrations caused by the breakers and foam lines. It would be expected then that a reasonably high degree of compaction would be attained. In view of the importance of compaction both in increasing the bearing capacity of a sand and in preventing the danger of liquefaction it would seem desirable that a study of this factor on various beaches should be made.

10. Effect of Cusps, Scarps, and Depressions on Trafficability. (VB-15)

Custs prove a serious hazard to the trafficability of beaches. On any beach they necessitate a substantial reduction in the speed at which a vehicle can be driven parallel to the beach and may cause a vehicle to get stuck when heading up a ridge of a cusp. A jeep which got stuck heading up the ridge of a cusp on Soldiers Club Beach, Fort Ord, Monterey, California is shown in Figure VB-16a. In addition to their direct affect on trafficability, cusps also cause currents in the surf zone which affect landings.

During periods when high steep "storm" waves cut back the beach face, scarps are formed. When they are high their adverse effect on trafficability is quite apparent. However, even small scarps, only a few feet in height, may cause considerable hazards to an amphibious operation. For example, at Santa Margarita River Beach, Oceanside, California, an attempt was made to drive an LVT (3) (c) up a two foot vertical scarp at an angle. As a result a track was thrown due to

lateral slippage (see Figure VB-16d). After the track had been replaced three further attempts were made to drive the tractor up the scarp, at right angles to the scarp, which appears to be the best way to overcome such an obstacle. The tractor succeeded at the third attempt and each time the large area of its relatively unprotected front hull was exposed to what would have been enemy fire on a defended beach (Figure VB-16c).

Occasionally a depression will occur in a beach, perhaps due to scouring around a landing craft at high tide and the scoured zone becoming exposed at low tide. Such depressions have proved to be hazardous to vehicles travelling along the beach. For example, at Santa Margarita River Beach, Oceanside, California a DUKW became stuck in a hole caused by an LCVP during landing (see Figure VB-16b). A constant guard should be maintained against hazards of this kind.

11. Effect of Backshore Conditions on Trafficability. (VB-15)

The backshore conditions of a beach may vary between extremely wide limits, sand dunes, sloughs, cliffs and wooded areas being typical examples of the conditions which may be encountered behind a beach. Even sand dunes, which are somewhat similar in character to the beach itself, are not easily negotiable, and tractors which have successfully traversed the beach have been found on occasions to become stuck in the backshore dunes. At Soldiers Club Beach, Fort Ord, Monterey, California an LVT ran parallel to the beach in the wave uprush in low range and ran with ease on the dry berm in low range in second gear. Nevertheless, it had great difficulty ascending the sand dunes and eventually became stuck when it threw a track because of lateral slippage (see Figure VB-17A). Again at Clatsop Spit Beach, Fort Stevens, Oregon, an LVT could operate with ease on the beach but became stuck on the dunes backshore (see Figure VB-17B).

Because of their varied nature it is impossible to discuss the general trafficability of backshore conditions. However, their characteristics are of just as much importance in determining the success of an amphibious operation as the trafficability characteristics of the beach itself.

12. Vehicle Characteristics Affecting Trafficability

A summary of some of the more important vehicle characteristics affecting the trafficability of soils has been presented by R. C. Stewart and S. J. Weiss (VB-9). The following review is taken mainly from their report.

The main factors affecting the performance of a vehicle on a given soil appear to be:

a. Nominal or Mean Pressure Between Track or Tire and Ground

This has a definite but as yet undetermined effect on actual pressures which exist under the tires or tracks. The greater the nominal pressure the greater is the bearing capacity required to support the vehicle and the greater is the rolling resistance. However, the greater the nominal pressure the greater also is the weight available to develop tractive force by friction between the tire or track and the soil. Hence, the variation of nominal pressure for a given vehicle on a given soil results in maximum tractive effeciency (pull/weight basis) being developed at some optimum loading. The optimum loading increases as the soil conditions improve.

b. Load Distribution

The distribution of vehicle load to the wheels or tracks is affected by the number, spacing and diameter of the wheels of tracks, by the soil conditions and by the position of the centre of gravity of the load. Advantage seems to be with a somewhat forward centre of gravity. A good loading distribution results in a minimum of sinkage and external rolling resistance.

Under each individual tire or track there are local pressure maxima and minima which vary with the tire or track and the soil conditions.

c. Width of Track or Tire

For a given nominal unit pressure on a cohesionless soil there is an optimum width of loaded area for which settlement is a minimum. For area widths below the optimum, settlement is greater than the minimum due to plastic flow of the soil and for area widths greater than the optimum, settlement exceeds the minimum due to greater elastic compression of the soil. The greater the settlement or sinkage, the greater is the rolling resistance.

For a given nominal pressure on a cohesive soil, settlement or sinkage increases with the width of the loaded area. On cohesive soils, traction also increases with the track or tire width.

An increase in the width of the tracks or tires of a given vehicle reduces the nominal pressure and therefore the bearing capacity required to support the vehicle. For cohesionless soils the bearing capacity also increases with the width of the loaded area so that an increase in track or tire width causes a marked reduction in sinkage and rolling resistance.

Track extensions and attachments, designed to increase flotation, are used to improve the trafficability of vehicles on soft muds. In general, little effect is obtained from flotation extensions without longer grousers to increase the tractive force. Little work can be performed with such extensions but the vehicle can move itself. The value of the extensions and attachments may be considerably reduced if the time required to fit them is excessive or if they cause undesirable road damage.

d. Grouser Dimensions and Spacing

For cohesionless soils and given loading conditions there are apparently optimum dimensions and proportions of track plates or grousers at which traction is a maximum. An increase in grouser length causes an increase in tractive force and also an increase in rolling resistance. Therefore, a compromise is necessary since the efficiency depends on tractive force minus rolling resistance. Excessive grousers lead to inefficiency.

In muddy conditions the virtue of track width appears to lie largely in the length of grouser which the track width affords. Increases of track width without corresponding increase in grouser length may be of little or no advantage. Laboratory experiments indicate that increases in grouser length give proportional increases in traction but that increases in track width alone gives a much smaller increase in traction.

Experiments (VB-9) have shown that plain or slightly rounded track

plates without grousers give the best performance in turf overlying deep mud or in soft snow. Designing the thrust faces of the grouser obliquely to the vertical (angled grouser) also appears to have some advantage in all soils except hard plastic clays where the effect is the same for any grouser. A track plate having adjustable grouser angles should be adjusted to the optimum conditions.

Successful experiments with certain rigid types of tracks fitted over the rear pairs of wheels give promise of a method whereby wheeled vehicles can be converted into semi-tracked vehicles and then successfully negotiate swampy conditions. This may also be applicable to amphibious vehicles experiencing difficulty in developing traction during the transition from water to soil.

e. Wheel Diameter

Rolling resistance decreases with increase in wheel diameter. Furthermore owing to the greater area of contact between the wheel and the ground, on a cohesive soil traction also increases with wheel diameter.

f. Type of Tire

Due to their lower contact pressure and greater contact area, pneumatic tires develop less rolling resistance than do solid tires.

g. Suspension System

On sticky soils, the soil adheres to the wheel or track and accumulates on the track rollers or beneath the fenders or carriage. This mud-packing increases the rolling resistance and may lead to power-stalling if the suspension system is too complex. The simpler the suspension system, consistant with proper load distribution, the better the results in such soils.

h. Ground Clearance

On soft muds a vehicle may sink until the undercarriage is resting on the ground surface. This causes an increase in rolling resistance and a decrease in traction. In such cases, the smaller the ground clearance of the vehicle, the sooner the vehicle tends to be immoblized.

13. Effect of Beach Face Slope on Trafficability

a. Effect of Beach Face Slope on Soil Characteristics

Investigations of many beaches along the Pacific coast of the United States and on Oahu, T. H. (VB-15) have shown a remarkably consistent relationship between the slope of the beach face and the size and shape of the grains comprising the beach materials.

Laboratory work in England (VB-1) has shown that the slope of a beach face depends largely on the material and is independent of the breaker height or tide stage during equilibrium conditions. Further studies at the University of California (VB-2) have indicated that the slope is somewhat dependent on the wave steepness, with flat waves increasing the slope to some extent and steep waves reducing the slope. Nevertheless, the type of material comprising the beach appears to be a major factor in determining the slope.

It would be expected that the properties of the beach material which

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are most effective in determining the beach slope would be the grain size, the grain shape, the grading of the grains, the specific gravity of the grains and the degree of compaction of the material. Most beach materials studied in the University of California investigations consist of grains which are relatively uniform in size and have a specific gravity of about 2.65. Hence, the influence of these factors can be considered the same for the majority of beaches. However, their effects may be significant in special cases and the use of correlations between beach slope, grain size and grain shape, established from investigations of the average type of beach, should be used with caution when applied to beaches where the grading of the grains or the specific gravity of the grains differ considerably from their usual values.

No data are available to indicate the effect of the degree of compaction of the beach material on the slopes of different beaches. However, data on beach face slope, grain size, grain shape and grading of the beach material for the eight beaches at which trafficability studies were made by the University of California are presented in Table VB-4.

It will be seen that for these beaches the coefficient of uniformity of the grains is remarkably constant, varying between 1.3 and 3.0 with an average value of 1.8. Such low values indicate materials in which the grains are relatively uniform in size, a characteristic of beach sands.

The data in the table also show that in general, the steeper the beach face, the larger and more rounded are the sand grains. The roundness and sphericity both decrease consistently with decreasing beach slope and the visual classification of the grain shapes shows increasing angularity with decreasing slope. This relationship may also be seen in Figure VB-18 in which photomicrographs of the sand samples are arranged in order of beach slope from the steepest beach to the flattest beach studied. In addition, it may be seen from this figure, from Table VB-4 and from Figure VB-19, that both the median sand diameter and the effective size show a general decrease with decreasing steepness of the beach face. This relationship was also observed in previous investigations of beach forms and materials along the Pacific coast of the United States carried out by the University of California (VB-2). The correlation between beach slope and grain size is shown by Figure VB-20. The data plotted in this figure are for the beaches listed in the table, on which trafficability studies were made, and for other beaches on which only the beach form and material were investigated.

The influence of these conditions on the trafficability of beaches is due to two main effects:

(i) For sands of the same density and uniformity, the more angular the grains, the greater is the angle of internal friction. Hence, the greater is the bearing capacity of the sand. Also, the more angular the grains, the greater is the traction of vehicle tires on the sand. Thus, the steeper beaches having the more rounded grains would tend to be the less trafficable.

(ii) For sands having the same uniformity and density, the smaller the grains, the smaller are the void diameters and therefore the greater is the curvature of the water menisci which form when the sand is moist or partially saturated. Hence, the greater is the apparent cohesion of the sand and the greater is the bearing capacity when moist. Thus, the steeper beaches having the larger grains would again tend to be the less trafficable.

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Table VB-4

| Beach | Average Slope Beach Face | Median Sand Diameter mm. | Average Effective size mm. | Coeff. of Uniformity | Roundness | Sphericity | Grain shape |
|---|---|---|---|---|---|---|---|
| Operation MIKI Beaches, Oahu, T.H. | 1/6.0 or $9\frac{1}{2}^{o}$ | 0.55-0.75 (0.65) | 0.46 | 1.45-1.8 (1.6) | 0.8 | 0.85 | Rounded and subrounded |
| Soldiers Club Beach Monterey, California | 1/7.8 or $7\frac{1}{4}^{o}$ | 0.6-0.75 (0.67) | 0.38 | 1.8-3.0 (2.3) | 0.5-0.8 | 0.85 | Subrounded and subangular |
| Point Reyes Beach, California | 1/9.6 or $6^{o}$ | 0.78-0.84 (0.81) | 0.61 | 1.4-2.0 (1.8) | 0.8 | 0.8 | Rounded and subrounded |
| Tank Farm Beach Monterey, California | 1/11.2 or $5^{o}$ | 0.44-0.56 (0.5) | 0.31 | 1.55-1.8 (1.7) | 0.7 | 0.75 | Subrounded and subangular |
| South Beach Moss Landing, Calif. | 1/17.8 or $3\frac{1}{4}^{o}$ | 0.23-0.3 (0.27) | 0.20 | 1.5-2.4 (1.7) | 0.4 | 0.7 | Subrounded and subangular |
| Cobble Beach Monterey, California | 1/19 or $3^{o}$ | 0.28-0.29 (0.28) | 0.23 | 1.3-1.4 (1.4) | 0.55 | 0.75 | Subangular, small proportion, subrounded |
| Santa Margarita River Beach, Oceanside, Cal. | 1/23 or $2\frac{1}{2}^{o}$ | 0.21-0.28 (0.25) | 0.17 | 1.55-1.9 (1.7) | 0.4 | 0.6 | Subangular |
| Clatsop Spit, Oregon | 1/62 or $1^{o}$ | 0.17-0.24 (p.21) | 0.15 | 1.45-1.6 (1.5) | 0.4-0.5 | 0.6 | Subangular |

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Each of these effects, together with the direct effect of slope on rolling resistance, indicates that the flatter a beach, the more trafficable it will be. This conclusion is confirmed by the data presented in Figure VB-15, which shows the maximum velocity at which DUKWs were able to travel on beaches of various slopes. Additionally, it will be seen that for a beach of a given slope, a DUKW can attain a higher speed when the sand is wet than if the sand is dry; also, for wet or for dry sand, the maximum DUKW velocity increases with decreasing steepness of the beach face.

b. Direct Effect of Slope on Trafficability

The maximum force which a vehicle can develop in addition to moving itself is called the drawbar pull of the vehicle. From a knowledge of the drawbar pull and the rolling resistance of the vehicle on level ground, together with the adhesion characteristics of the soil, or the drawbar pull and the coefficient of friction between the wheel and the soil, it is possible to determine:

(i) the maximum grade on which the vehicle can operate

(ii) the maximum gross load the vehicle can pull up a given slope.

The analyses of these conditions are presented below:

(1) Maximum Grade on Which Vehicle can Operate When Limited by Slippage: Figure VB-21 shows a wheel resting on level ground. On hard dry ground the tractive force F is developed by friction between the wheel and the ground and may be expressed by the formula:

$$F = N \cdot f$$

where f = coefficient of friction

F = tractive force

N = W = force normal to ground.

On soft muddy soils the tractive force is not developed only by friction but is partly due to adhesion between the wheel and the soil. The magnitude of the adhesion component depends on the area of wheel in contact with the soil. However, for a vehicle of given weight the area of contact between the tire and the soil will not vary greatly whether the vehicle is on a slope or on level ground and in this analysis will therefore be assumed constant. Hence, in general, the tractive force F may be expressed as follows:

$$F = A + Nf$$

where A is the adhesion between the wheel and the soil.

Provided the vehicle has sufficient power to cause slippage, the drawbar pull does not depend directly on the horsepower of the vehicle since the only force effective in moving the wheel forward is the traction between the wheel and the ground. Thus for a wheel on level ground:

$$\text{Maximum tractive force } F = A + Wf$$



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Rolling resistance = R

Maximum drawbar pull = $P_1 = F - R = A + Wf - R$

where $P_1$ = pull necessary to cause slippage, and W = weight on tractive wheels

hence $R = A + Wf - P_1$.

If the same vehicle is ascending a slope with gradient $\alpha$ as shown in Figure VB-22, it can be seen from the force diagram that the force normal to the ground is $W \cos \alpha$ and an additional force $W \sin \alpha$ is resisting forward motion of the wheel. Hence:

Maximum tractive force = $A + W \cos \alpha \cdot f$

Resistance to motion = $R + W \sin \alpha$.

$\therefore$ Net drawbar pull = $A + Wf \cos \alpha - R - W \sin \alpha$.

The maximum slope the vehicle can ascend is one for which the uphill force just balances the downhill force and the net drawbar pull which could be developed is equal to zero. Thus if $\alpha_{max}$ denotes the maximum slope the vehicle can ascend:

$$A + Wf \cos \alpha_{max} - R - W \sin \alpha_{max} = 0.$$

Substituting for R from the above equation:

$$A + Wf \cos \alpha_{max} - (A + Wf - P_1) - W \sin \alpha_{max} = 0$$

$$\therefore \tan \alpha_{max} = f - \frac{f}{\cos \alpha_{max}} + \frac{P_1}{W} \cdot \frac{1}{\cos \alpha_{max}}$$

Now $$\frac{1}{\cos \alpha_{max}} = \frac{1}{1 - (1 - \cos \alpha_{max})}$$

$$= \left[1 - (1 - \cos \alpha_{max})\right]^{-1}$$

$$\approx 2 - \cos \alpha_{max}, \text{ if } \alpha_{max} \text{ is small.}$$

In most cases the maximum slope a vehicle can ascend is relatively flat. Hence,

$$\tan \alpha_{max} = f - f\,(2 - \cos \alpha_{max}) + \frac{P_1}{W}\,(2 - \cos \alpha_{max})$$

$$= \frac{P}{W} - \left(f - \frac{P_1}{W}\right)(1 - \cos \alpha_{max})$$

An alternative form of this expression is:

$$\tan \alpha_{max} = \frac{P_1}{W} - \frac{(R - A)}{W}\,(1 - \cos \alpha_{max})$$

On a sandy soil, A = 0 and if R is also equal to zero or is very small then

$$\tan \alpha_{max} = \frac{P_1}{W}$$

Again, on a cohesive soil for which R = A,

$$\tan \alpha_{max} = \frac{P_1}{W}$$

On soils for which R > A the term $\frac{(R-A)}{W}$ $(1 - \cos \alpha_{max})$ will always be positive and therefore

$$\tan \alpha_{max} < \frac{P_1}{W}$$

Thus the ratio $\frac{P_1}{W}$ is in many cases indicative of the tangent of the angle of maximum slope which a vehicle can ascend. If the coefficient of friction f is also known, or alternatively, if the rolling resistance and adhesion are known, a close estimate of the maximum slope can be made from the above formula.

Computations were made by the Corps of Engineering (VB-12) to determine the effect of the change in load on the axles caused by the shift in centre of gravity when a truck is on a slope. It was found that for trucks with all-wheeled drive there is no change. For trucks with drive on the rear axle only the effect is not appreciable where the height to the centre of gravity of the load is less than 1/5 of the wheel base. For the extreme case of a 45° slope, the increase in load on the rear axle was only 5%.

(2) Determination of Towing Capacity when Limited by Slippage

Let $P_1$ = maximum drawbar pull of towing vehicle on level ground.

$\alpha$ = angle of slope

$W_1$ = weight of towing vehicle

$W_2$ = weight of towed vehicle

$A_1$ = adhesion for towing vehicle

$A_2$ = adhesion for towed vehicle

$f_1$ = coefficient of friction for towing vehicle

$f_2$ = coefficient of friction for towed vehicle

$R_1$ = rolling resistance for towing vehicle

$R_2$ = rolling resistance for towed vehicle

$R_{2s}$ = sliding resistance of towed vehicle other than friction and adhesion (assumed constant)

The force diagram is shown in Figure VB-23. It is assumed that the drawbar pull of the towing vehicle is limited, not by power, but by slippage of the wheels.

(a) Case 1. When Towed Vehicle is Slipping:

The tractive force of the towing vehicle up the slope = $A_1 + W_1 \cos\alpha f_1$

The resistance to motion of the towing vehicle = $R_1 + W_1 \sin\alpha$

$\therefore$ Net drawbar pull of towing vehicle = $A_1 + W_1 f_1 \cos\alpha - R_1 - W_1 \sin\alpha$

Resistance to motion of towed vehicle = $A_2 + W_2 f_2 \cos\alpha + R_{2s} + W_2 \sin\alpha$

When slippage occurs for the towing vehicle, the resistance to motion of the towed vehicle will just be equal to the net drawbar pull. If $W_{2_{max}}$ is the weight of the towed vehicle when this occurs and $R_{2s_{max}}$ is the corresponding sliding resistance:

$$A_1 + W_1 f_1 \cos\alpha - R_1 - W_1 \sin\alpha = A_2 + W_{2_{max}} f_2 \cos\alpha + R_{2s_{max}} + W_{2_{max}} \sin\alpha$$

$$\therefore W_{2_{max}} (f_2 \cos\alpha + \sin\alpha) = W_1 (f_1 \cos\alpha - \sin\alpha) + A_1 - A_2 - R_1 - R_{2s_{max}}$$

$$\therefore W_{2_{max}} = \frac{W_1 (f_1 \cos\alpha - \sin\alpha) + A_1 - A_2 - R_1 - R_{2s_{max}}}{f_2 \cos\alpha + \sin\alpha}$$

Assuming the rolling resistance and adhesion of the towing vehicle are the same on the slope as on level ground,

$$P_1 = A_1 + W_1 f_1 - R_1$$

$$\therefore A_1 - R_1 = P_1 - W_1 f_1$$

$$\text{Hence } W_{2_{max}} = \frac{P_1 - W_1 (f_1 + \sin\alpha - f_1 \cos\alpha) - A_2 - R_{2s_{max}}}{f_2 \cos\alpha + \sin\alpha}$$

An alternative form of this expression is

$$W_{2_{max}} = \frac{P_1 \cos\alpha - W_1 \sin\alpha + (A_1 - R_1)(1 - \cos\alpha) - A_2 - R_{2s_{max}}}{f_2 \cos\alpha + \sin\alpha}$$

On a sandy soil $A_1 = 0$ and $A_2 = 0$. If $R_1$ and $R_{2s_{max}}$ are also negligible

$$W_{2_{max}} = \frac{P_1 \cos\alpha - W_1 \sin\alpha}{f_2 \cos\alpha + \sin\alpha} = \frac{P_1 - W_1 \tan\alpha}{f_2 + \tan\alpha}$$

If only $R_{2s_{max}}$ is negligible, then

$$W_{2_{max}} = \frac{P_1 - W_1 (f_1 + \sin\alpha - f_1 \cos\alpha)}{f_2 \cos\alpha + \sin\alpha}$$

On level ground $\alpha = 0$

$$\therefore W_{2_{max}} = \frac{P_1 - A_2 - R_{2s_{max}}}{f_2}$$

On a sandy soil $A_2 = 0$ and if $R_{2s_{max}}$ is negligible,

$$W_{2_{max}} = \frac{P_1}{f_2}$$

From the above expressions the towing capacity of a vehicle can be determined if the requisite factors are known.

Maximum Slope for Towing Vehicle Only: The above formulas can also be used to determine the maximum gradient which the towing vehicle itself can ascend. This is found by equating $W_{2_{max}}$ to zero. This means that the towing vehicle is just able to ascend the slope while pulling zero load and the corresponding value of $\alpha$ is therefore the limiting slope. Thus, if

$$W_{2_{max}} = 0,\ R_{2s_{max}} = 0,\ A_2 = 0 \text{ and}$$

$$P_1 \cos \alpha_{max} - W_1 \sin \alpha_{max} + (A_1 - R_1)(1 - \cos \alpha_{max}) = 0.$$

$$\therefore \tan \alpha_{max} = \frac{P_1}{W_1} - \frac{R_1 - A_1}{W_1} \cdot \frac{1 - \cos \alpha_{max}}{\cos \alpha_{max}}$$

$$= \frac{P_1}{W_1} - \frac{R_1 - A_1}{W_1} \left( \frac{1}{\cos \alpha_{max}} - 1 \right)$$

Writing $\frac{1}{\cos \alpha_{max}} = 2 - \cos \alpha_{max}$ since $\alpha_{max}$ is small.

$$\tan \alpha_{max} = \frac{P_1}{W_1} - \frac{R_1 - A_1}{W_1} (1 - \cos \alpha_{max})$$

This is the same expression as that previously obtained.

Maximum Slope for Towing and Towed Vehicles: The maximum gradient which both towing and towed vehicles can ascend together may be determined as follows:

Drawbar pull of towing vehicle $= A_1 + W_1 \cos \alpha \cdot f_1 - R_1 - W_1 \sin \alpha$

Resistance of towed vehicle (assuming slippage) =

$$A_2 + W_2 \cos \alpha \cdot f_2 + R_{2s} + W_2 \sin \alpha$$

The maximum slope which can be ascended is the value of $\alpha$ when the net drawbar pull is zero. If this slope is $\alpha_{max}$ then:

$$A_1 + W_1 f_1 \cos \alpha_{max} - R_1 - W_1 \sin \alpha_{max} - A_2 - W_2 f_2 \cos \alpha_{max} - R_{2s} - W_2 \sin \alpha_{max} = 0$$

$$\therefore \tan \alpha_{max} = \frac{W_1 f_1 - W_2 f_2}{W_1 + W_2} - \frac{R_1 + R_{2s} - A_1 + A_2}{W_1 + W_2} \cdot \frac{1}{\cos \alpha_{max}}$$

But $P_1 = A_1 + W_1 f_1 - R_1$ and writing $\frac{1}{\cos \alpha_{max}} = 2 - \cos \alpha_{max}$

$$\tan \alpha_{max} = \frac{W_1 f_1 - W_2 f_2}{W_1 + W_2} - \frac{W_1 f_1 - P_1 + R_{2s} + A_2}{W_1 + W_2} (2 - \cos \alpha_{max}).$$

$$= \frac{P_1 - A_2 - R_{2s}}{W_1 + W_2} (2 - \cos \alpha_{max}) - \frac{W_1 f_1}{W_1 + W_2} (1 - \cos \alpha_{max}) - \frac{W_2 f_2}{W_1 + W_2}$$

Alternatively, $\tan \alpha_{max} = \frac{P_1 - W_2 f_2}{W_1 + W_2} - \frac{R_1 - A_1}{W_1 + W_2} (1 - \cos \alpha_{max}) - \frac{R_{2s} + A_2}{W_1 + W_2} (2 - \cos \alpha_{max})$

On a sand, $A_1$ and $A_2$ are both zero and if $R_{2s}$ is negligible,

$$\tan \alpha_{max} = \frac{P_1 - W_2 f_2}{W_1 + W_2} - \frac{R_1}{W_1 + W_2} (1 - \cos \alpha_{max})$$

(b) Case 2. When Towed Vehicle Rolls Without Slipping. If the towed vehicle rolls without slipping, the resistance to motion of this vehicle is $R_2 + W_2 \sin \alpha$.

The drawbar pull of the towing vehicle is $A_1 + W_1 f_1 \cos \alpha - R_1 - W_1 \sin \alpha$

If $W_{2_{max}}$ is the weight of the towed vehicle when slippage occurs for the towing vehicle then:

$$A_1 + W_1 f_1 \cos \alpha - R_1 - W_1 \sin \alpha = R_2 + W_{2_{max}} \sin \alpha$$

Also, $P_1 = A_1 + W_1 f_1 - R_1$

$$\therefore P_1 - W_1 (f_1 + \sin \alpha - f_1 \cos \alpha) = R_2 + W_{2_{max}} \sin \alpha$$

Since $R_2$ depends on $W_2$, this equation cannot be simplified unless this relationship is known, at least approximately.

The maximum slope which the two vehicles together can ascend is readily determined as before. In this case the net drawbar pull is

$$A_1 + W_1 f_1 \cos \alpha - R_1 - W_1 \sin \alpha - R_2 - W_2 \sin \alpha$$

and the maximum slope which can be ascended is $\alpha_{max}$ where

$$A_1 + W_1 f_1 \cos \alpha_{max} - R_1 - W_1 \sin \alpha_{max} - R_2 - W_2 \sin \alpha_{max} = 0$$

Simplifying this expression as before, leads to the equations:

$$\tan \alpha_{max} = \frac{P_1 - R_2}{W_1 + W_2} (2 - \cos \alpha_{max}) - \frac{W_1 f_1}{W_1 + W_2} (1 - \cos \alpha_{max})$$

$$\text{or } \tan \alpha_{max} = \frac{P_1}{W_1 + W_2} - \frac{R_1 - A_1}{W_1 + W_2} (1 - \cos \alpha_{max}) - \frac{R_2}{W_1 + W_2} (2 - \cos \alpha_{max})$$

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On a sand, $A_1$ is zero and if $R_1$ and $R_2$ are very small,

$$\tan \alpha_{max} = \frac{P_1}{W_1 + W_2}$$

In general, $\tan \alpha_{max}$ will be less than $\frac{P_1}{W_1 + W_2}$

(3) Field Measurements of Effect of Gradient: Tests were conducted by the Corps of Engineers (VB-12) in which the drawbar pulls developed by a vehicle on a soft silty clay soil on level ground and on $3\frac{1}{2}\%$ and 7% slopes were measured. The drawbar pulls developed on the slopes were compared with those calculated from the formulas presented above, but neglecting the effects of rolling resistance and adhesion. It was found that the drawbar pulls computed in this way were in fair agreement with the measured values.

14. Relative Bearing Capacities Under Static Loads of Sections of Sandy Beach

From the bearing capacity formulas previously described it is possible to estimate the relative bearing capacities of the various sections of a sandy beach. The formulas are known to be conservative for sands but the degree of inaccuracy will be the same for all sections considered and the use of the formulas will therefore have no appreciable effect on the relative bearing capacities of the various beach sections.

In a consideration of bearing capacity it is convenient to divide a beach into four main sections where the sand is either dry, moist, saturated or submerged. The ultimate bearing capacity of these sections under static strip loads may be determined from the Terzaghi formula (VB-11):

$$q_u = cN_c + \tfrac{1}{2}\gamma' bN_\gamma + \gamma' dN_q$$

where $q_u$ = ultimate bearing capacity

$c$ = effective cohesion of soil

$\gamma'$ = effective unit weight of soil

$b$ = width of loaded area

$d$ = depth of loaded area below adjacent soil surfaces

$N_c$, $N_q$, and $N_\gamma$ are coefficients depending only on the effective angle of friction of the soil, $\phi$.

For a traffic load on the surface of the soil, the formula reduces to:

$$q_u = cN_c + \tfrac{1}{2} b \gamma' N_\gamma$$

Hence, for the same vehicle, or the same width of loaded area, the relative bearing capacities under static loading conditions of the dry, moist, saturated and submerged sections of a beach depend on the relative magnitudes of the effective cohesion, $c$, the effective unit weight, $\gamma'$, and the effective angle of



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friction, $\phi$, in the various sections. The values of c, $\gamma$ and $\phi$ will vary as follows:

(i) Cohesion: The true cohesion of a sand is zero. However, the moist section of beach will have apparent cohesion due to the menisci effects previously described. In saturated sand the apparent cohesion will be very small, while dry or submerged sand, in which there are no water menisci, will have no apparent cohesion. The smaller the effective grain size of the sand the greater will be the apparent cohesion.

(ii) Unit Weight: The effective unit weight of the sand in different sections of a beach will vary due to the effect of compaction and submergence. The degree of compaction of those sections of a beach submerged at high tide is probably greater than for the dry section due to the compactive effect of submergence itself and of the vibrations caused by breakers. In addition, for a given void ratio, the greater the proportion of the voids filled with water, the higher is the unit weight of a soil. Hence, the unit weight of the moist section will be greater than that of the dry section and will increase with increasing water content, reaching a maximum in the saturated section. However, in the submerged section, the buoyancy effect of the water causes a reduction in the effective weight of the soil grains and the unit weight of a submerged sand is only about one-half that of the saturated sand above the water table.

(iii) Angle of Friction: For a given sand, the angle of friction depends mainly on the void ratio. The angle of friction of a beach sand will be approximately the same for the moist, saturated and submerged sections of a beach but will probably be slightly lower in the dry section where the sand is probably in a more loose condition.

The variation of cohesion, unit weight and angle of friction of the sand in different sections of a beach is summarized in Table VB-5. From a consideration of this table and the Terzaghi formula it may be seen that in order of decreasing bearing capacity for static loads, the different sections of a beach would be arranged as follows:

Moist - bearing capacity varying between about $1.8q_{ud}$ and $q_{ud}$ depending on water content.

Saturated - bearing capacity varying between about $1.6q_{ud}$ and $0.8q_{ud}$ depending on position of water table.

Dry - bearing capacity = $q_{ud}$.

Submerged - bearing capacity varying between about 0.8 and $0.9q_{ud}$.

Table VB-5

Variation of Cohesion, Unit Weight and Angle of Friction Across Beach

| Section of Beach | Cohesion | Unit Weight | Angle of Friction |
|---|---|---|---|
| Dry | Zero | Relatively loose condition. Say value $w_d$. | Relatively loose condition. Say value $\phi_d$. |
| Moist | Apparent cohesion | Greater than $w_d$ due to compaction and water content. Probably value about 1-1 to $1.5w_d$. | Probably slightly greater than $\phi_d$ due to lower void ratio |
| Saturated | Little apparent cohesion | Maximum value for moist sand. Probably value about $1.5w_d$. | Probably slightly greater than $\phi_d$ due to lower void ratio |
| Submerged | Zero | Less than unit weight of saturated sand due to buoyancy effects. Probably value about $0.7w_d$. | Probably slightly greater than $\phi_d$ due to lower void ratio. |

Whether or not the bearing capacity of the moist section is greater or less than that of the saturated section will depend on the actual water content. Possible relative values of the bearing capacities of the different sections are also indicated above. However, it is emphasized that these values are merely meant to be indicative of the order of magnitude of the relative bearing capacities of the different sections and are not based on actual measurements. Nevertheless, it can be concluded that the bearing capacity for static loads will be a maximum somewhere in the moist section of a sandy beach and will decrease in the dry and submerged sections; also that the bearing capacity of the dry section will probably exceed that of the submerged section.

This type of distribution has been observed in beach studies conducted by the University of California (VB-15). Cone penetrometer measurements were made on many beaches and the variation of cone index across the profile of the beach was plotted. The cone index is a relative measure of the static bearing capacity of a soil. Hence, the variation in cone index across the profile is indicative of the variation in bearing capacity under static loads. Typical test results showing the variation in cone index across two profiles of Santa Margarita River Beach, Oceanside, California, and one across Clatsop Spit Beach, Oregon, are shown in Figures VB-24, 25 and 26. It will be seen that the maximum cone index occurs at a zone in the moist section of the beach which is nearer to the dry berm than to the water's edge; from this zone the cone index decreases towards the dry and submerged sections of the beach. The secondary peaks in the cone index distribution curve are in all probability due to minor variations in beach material across the profile.

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The variation of static bearing capacity is also indicated by the difficulty in starting a vehicle at the different sections. For example, at Tank Farm Beach a DUKW could not move after standing in the submerged section or on the dry berm but would start on the moist beach face. Similarly at South Beach, a jeep could start with ease on the saturated or moist sections but would not start up on the dry berm. These specific cases are typical of the conditions experienced at all beaches investigated.

15. Effect of Vehicle Speed on Trafficability of Sandy Beaches

An aspect of the trafficability of soils which has received little attention is the effect of transient loading on bearing capacity. Tests performed at Harvard University (VB-4) have shown that the strength of dry sand in transient tests with a time of loading of 0.02 seconds is about 20% greater than the strength of an identical specimen for which the time of loading is 10 minutes; for a time of loading of 0.2 seconds the strength is about 10% greater than the strength for a time of loading of 10 minutes. Transient loading has a much greater effect on the strength and modulus of deformation of clays. For a time of loading of 0.02 seconds the strengths of the clays tested were between 40% and 160% greater than their strengths for times of loading of 10 minutes; the corresponding increase in modulus of deformation was about 100%.

A time of loading of 0.02 seconds would correspond approximately to that produced by a wheeled vehicle travelling at about 30 m.p.h; a time of loading of 0.2 seconds would correspond approximately to a wheeled vehicle moving at 3 m.p.h. Hence, the greater the speed of a vehicle, the greater will be the bearing capacity of the soil and the smaller will be the deformation of the soil under load.

No data is available for the effect of transient loading on the strength and modulus of deformation of saturated sands. However, it seems possible that the increase in strength, due to transient loading of compact saturated sand will be greater, due to menisci effects than the increase in strength of dry sands. Further research on this point would be very desirable.

It is apparent, however, that under the transient effects of traffic loads the bearing capacity of a soil will be greater than for static loads. If, in addition, it is assumed as a tentative hypothesis pending the results of further investigations, that transient loading causes an increase of, say 50% in the strength and modulus of deformation of a compact, saturated sand above the static values, compared with a corresponding increase of about 20% for a dry sand (as measured in tests), the previous concept of the relative bearing capacities of the different sections of a beach would be considerably modified. Using the static bearing capacity values previously discussed, it will be seen that the bearing capacity for transient loads of the saturated section of a beach becomes about $2.4q_{ud}$, that for the submerged section becomes about $1.3q_{ud}$ while that for the dry section becomes only $1.2q_{ud}$. For the moist section, the bearing capacity for transient loads will decrease from about $2.4q_{ud}$ where the water content is high to about $1.2q_{ud}$ where the water content is very low. Hence, in order of decreasing bearing capacity for transient loads, the different sections of a beach would be arranged:

Saturated - bearing capacity between about 2.4 and $1.2q_{ud}$

Moist - bearing capacity between about 2.4 and $1.2q_{ud}$

Submerged - bearing capacity about $1.2q_{ud}$

Dry - bearing capacity about $1.2q_{ud}$

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Hence, the bearing capacity for vehicular traffic will be a maximum near the water's edge and will decrease progressively towards the backshore. Even in the submerged section, the bearing capacity will be about equal to that on the dry berm.

The increase of bearing capacity of a soil due to transient loads may be considered as an inertia effect. When a load, equal to the static bearing capacity, is applied to a soil, failure does not occur instantaneously but takes place over a short length of time. If the load is removed quickly the resultant effect will merely be a small deformation of the soil. However, if a load somewhat greater than the static bearing capacity is applied to the soil, failure will occur more rapidly and even if this load is again removed quickly, the deformation will be greater than in the previous case. As the magnitude of the transient load is increased, the magnitude of the resultant deformation will increase. Eventually a stage will be reached when the resultant deformation will be so large as to be termed failure. It is apparent that the transient load required to produce this deformation will be greater than the static bearing capacity of the soil and will increase as the time of application of the load decreases.

In the submerged section of a beach or in the zone between the limits of uprush and backrush, vehicle tracks caused perhaps by partial failure of the soil will tend to be eradicated by the movement of sand by the water. Hence, after each vehicle has passed, a new surface will tend to be prepared for the following vehicle. Beyond the zone of uprush, however, the deformation produced by a vehicle will remain. If this deformation is due to partial failure in the short length of time of application of the vehicle load, and following vehicles follow in the same tracks, the magnitude of the deformation will progressively increase until eventually the rolling resistance will increase to a point where the soil is no longer trafficable.

If the load due to a vehicle is less than the bearing capacity of the sand in the dry section of the beach, it will be less than the bearing capacity in either the moist or saturated sections of the beach and hence the magnitude of the deformations produced by the vehicle (the depth of the track or tire impression) in each section will depend on the degree of compaction and the modulus of deformation of the sand in that section. The impression caused by any vehicle will be due partly to compaction of the soil by the weight of the vehicle and the vibrations caused by the vehicle and partly to deformation of the soil under the applied loads. In the saturated and moist sections of a beach, the sand is probably more compact than in the dry section and hence the part of the tire or track impression which is due to compaction will be less than in the dry section. Furthermore, since the sand in the dry section of the beach is in a loose condition, the modulus of deformation under static loads will be less than in the moist and saturated sections and this modulus for dry sand will probably be less affected by transient loads. Hence, the depth of tire or track impressions on the dry section of a beach will be considerably greater than in the moist or saturated sections.

In the saturated section of the beach the actual depression under the wheel loads may be as great as in the moist section. However, in the saturated section the impression caused by the wheels will tend to be eliminated by movement of water and sand into the depressed zone and the residual impression will be much smaller than the actual depression, whereas in the moist section the full depression caused by the wheel loads will remain when the load is removed. It would be expected then that the residual depth of tire or track impression on the saturated section of the beach will be less than on the moist section of the beach and both will be less than on the dry section. In other words, the residual depth of track or tire impression will increase progressively as the vehicle moves from the saturated towards the dry section of the beach.

It need hardly be said that in discussing the bearing capacity of saturated sand it has been assumed that the sand is sufficiently compact that liquefaction will not occur.

It will be seen from the preceding discussion that vehicle speed affects trafficability in two ways; first through its effect on bearing capacity and deformations and second, through its effect in overcoming rolling resistance. The higher the speed of a vehicle, the more transient are the loads on the soil, the higher is the bearing capacity and the smaller are the depth of tire impressions in the soil. Hence, the lower is the rolling resistance and the better is the trafficability. The effect of depth of tire impression (which is related to the supporting capacity of a soil) on trafficability as measured by the maximum attainable speed of a vehicle on the soil is shown in Figure VB-27. It will be seen that the greater the depth of tire impression, the greater is the rolling resistance and the lower is the speed at which a DUKW can travel. Hence, the higher the velocity, the lower is the rolling resistance.

In addition to affecting the rolling resistance, the vehicle speed also helps to overcome the rolling resistance. The higher the initial speed, the greater is the momentum of the vehicle which is available to overcome the rolling resistance and the greater is the speed at which the vehicle can continue to move. Hence, higher vehicle speeds are beneficial in two ways; first in decreasing rolling resistance and secondly in providing more energy for overcoming rolling resistance. This combined effect is apparent in trafficability tests. For example, at Moss Landing South Beach a jeep could run with ease on the moist sand but had difficulty getting up the berm. If the jeep developed a fairly high velocity on the moist sand before reaching the berm it could successfully cross the berm; however, if the initial velocity was low the jeep would eventually be stopped by the berm sand and would be unable to start up again.

Thus in effect, vehicle speed affects supporting capacity which affects rolling resistance; vehicle speed also affects the forces available to overcome the rolling resistance. In turn, the rolling resistance affects the vehicle speed. This interdependence of rolling resistance and vehicle speed increases considerably the complexity of an analysis of the effects of vehicle speed on trafficability.

## 16. Effect of Density of Traffic on Trafficability

The density of traffic affects trafficability due to the effects of repeated loads on the soil. A given soil may permit only a limited number of passes of a given vehicle while it may permit an unlimited number of passes of a lighter vehicle.

There are several causes of the progressive reduction in trafficability of the soil in areas where only a limited number of passes can be made. In many cases, the depth of rut increases with the number of passes and thus the rolling resistance is progressively increased. Furthermore, the repeated deformation of the soil by the vehicle has a remoulding effect which tends to destroy any strength of a plastic soil which is due to its natural structure or to thixotropic properties of the soil. As the remoulding effect progresses, the soil conditions grow worse, the bearing capacity and traction decrease, the rolling resistance increases and eventually the vehicle may be immoblized.

Conditions have also been observed where repeated traffic loads on a firm

silty soil or fine sand, accompanied by a high water table, have had the effect of pumping water to the surface and causing the area to become impassable in a very short time.

Because of these effects it is desirable to specify the density of traffic to which an area will be subjected or for which an area is trafficable. The Corps of Engineers (VB-12) has proposed the use of two main groups:

(i) Limited operation - when the traffic density is not greater than about 25 repeated trips.

(ii) Unlimited operation - when the traffic density is greatly in excess of 25 repeated trips.

17. References

V B-1: Bagnold, Major R. A. - "Beach Formation by Waves; Some Model Experiments in a Tank" - Journal of the Institution of Civil Engineers, Vol. 15, No. 1, Paper no. 5237, November 1940, pp. 27-52.

V B-2: Bascom, W. N. - "The Relationship Between Sand Size and Beach Face Slope", Series 14, Issue 2, Institute of Engineering Research, University of California, Berkeley, California, April 1951 (Unpublished).

V B-3: Casagrande, Arthur - "Classification and Identification of Soils" - Proceedings, American Society of Civil Engineers, Vol. 73, No. 6, Part 1, June 1947, pp. 783-810.

V B-4: Casagrande, Arthur and Shannon, W. L. - "Research on Stress - Deformation and Strength Characteristics of Soils and Soft Rocks Under Transient Loading" - Soil Mechanics Series No. 31, Graduate School of Engineering, Harvard University, Cambridge, Mass.

V B-5: Krey, I. E. - Erddruck, Erdwiderstand Und Trägfahigkeit des Baugrundes", Berlin, 1932.

V B-6: Krynine, D. P. - "Soil Mechanics" - McGraw-Hill Book Co., New York, N. Y., 1941.

V B-7: McKibben, E. G. - "Some Kinematic and Dynamic Studies of Rigid Transport Wheels for Agricultural Equipment" - Ag. Expt. Station Bulletin, Iowa State College of Ag. and Mech. Arts.

V B-8: Prandtl, L. - "Härte Plashecher Körper" - Nach. Ges. Wiss., Göttingen, 1920.

V B-9: Stewart, R. C. and Weiss, S. J. - "Preliminary Study of the Mobility of Vehicles as Related to Trafficability of Traction Media" - Technical Memorandum M-003, U. S. Naval Civil Engineering Research and Evaluation Laboratory, Port Heuneme, California, 31 October 1950.

V B-10: Taylor, D. W. - "Fundamentals of Soil Mechanics" - John Wiley and Sons Inc., New York, N. Y., 1948.

V B-11: Terzaghi, K. - "Soil Mechanics in Engineering Practice" - John Wiley and Sons, Inc., New York, N. Y., 1948.

V B-12: Waterways Experiment Station - "Trafficability of Soils" - Waterways Experiment Station (U. S. Army, Corps of Engineers), Vicksburg, Miss. 1 Sept., 1945.

V B-13: Waterways Experiment Station - "Trafficability of Soils. Pilot Tests; Self-Propelled Vehicles" - Technical Memorandum Waterways Experiment Station (U. S. Army, Corps of Engineers), No. 3-240, Vicksburg, Miss., November 1947.

V B-14: Waterways Experiment Station - "Trafficability of Soils. Laboratory Tests to Determine Effects of Moisture Content and Density Variations", Technical Memorandum No. 3-240, First Supplement, Waterways Experiment Station (U. S. Army, Corps of Engineers), Vicksburg, Mississippi, March, 1948.

V B-15: Weigel, Robert L. - "Trafficability Studies of Selected Beaches of the Pacific Coast of the United States and Oahu, T. H." - Series 29 Issue 30, Institute of Engineering Research, University of California, Berkeley, California, 15 September 1950 (Unpublished) (Restricted).

"NONE"



"SOME"



"COMPLETE"



SPATULA STICKINESS

Figure V B-1

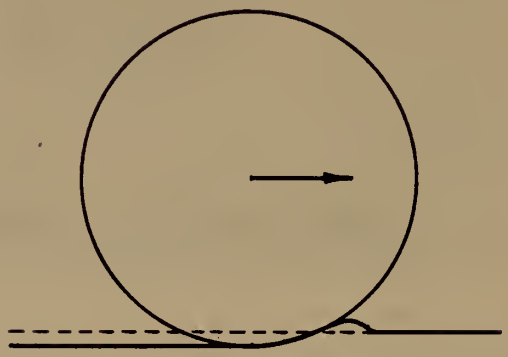


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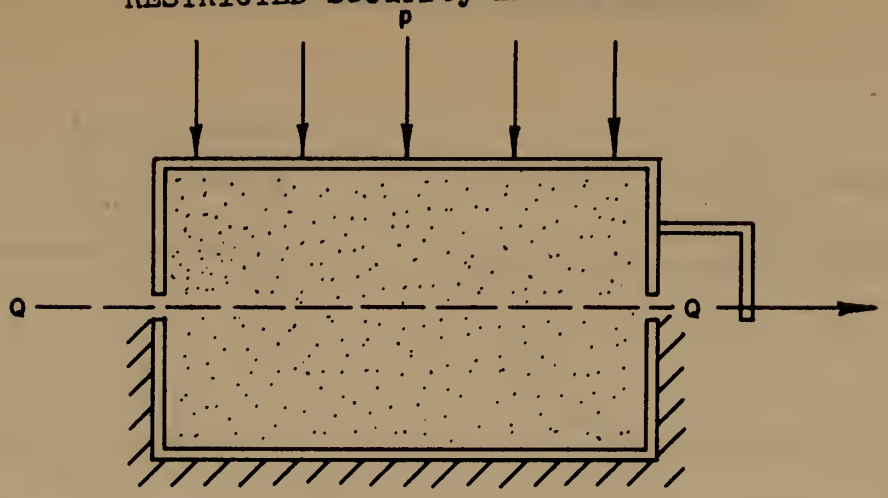



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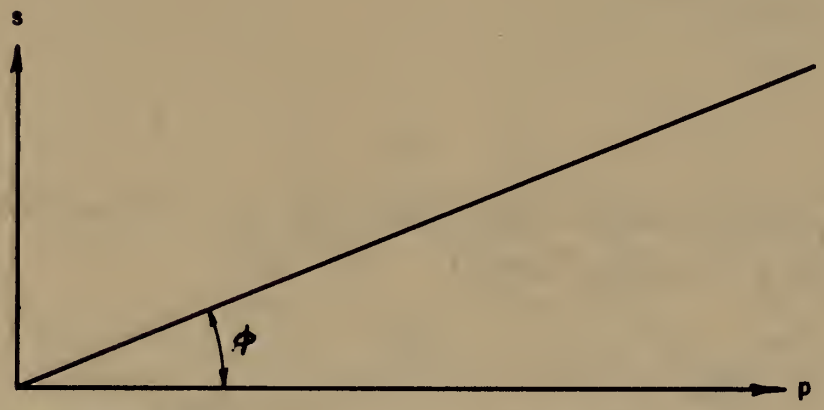



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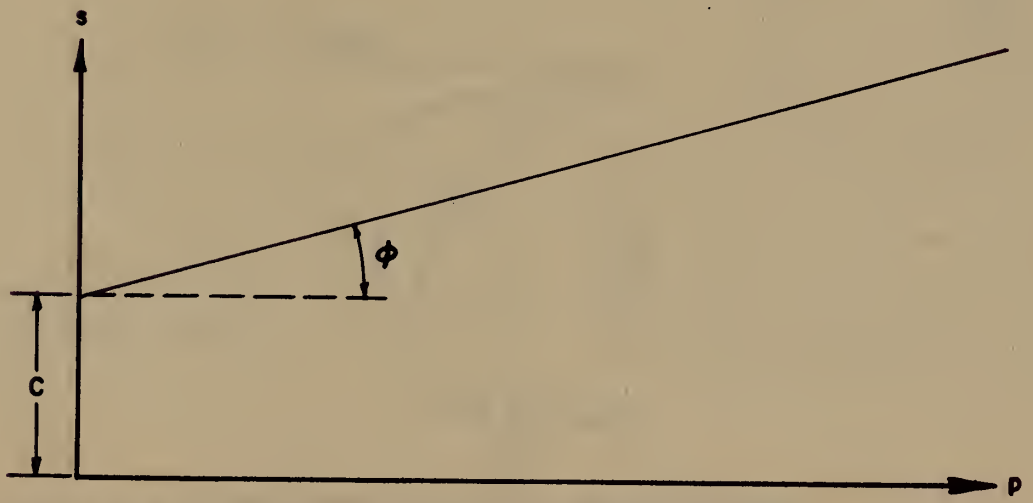



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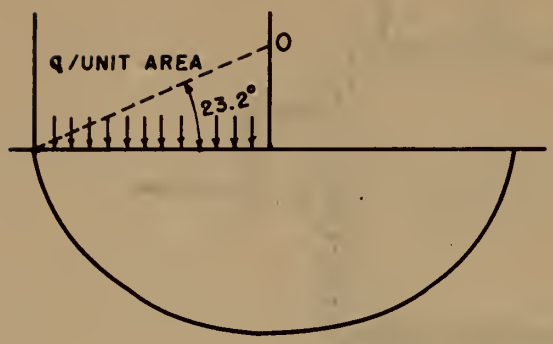



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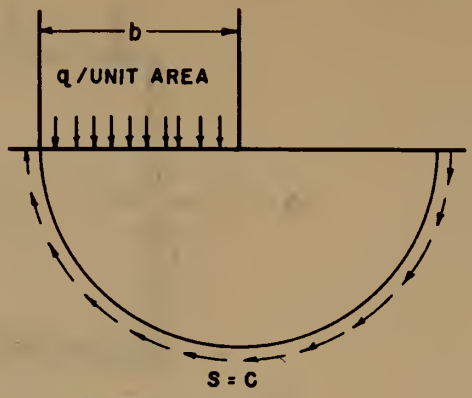



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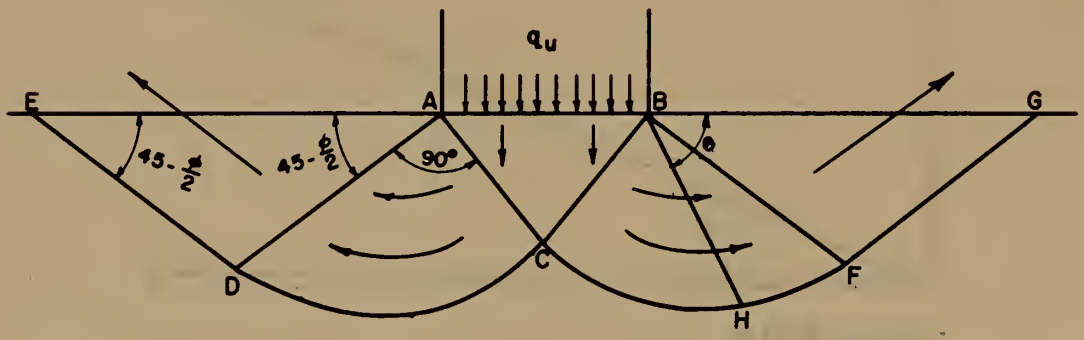



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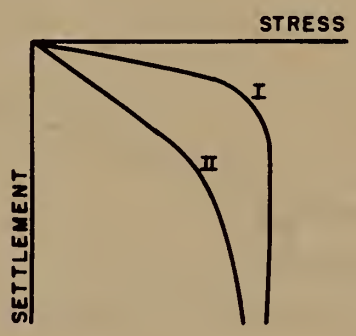



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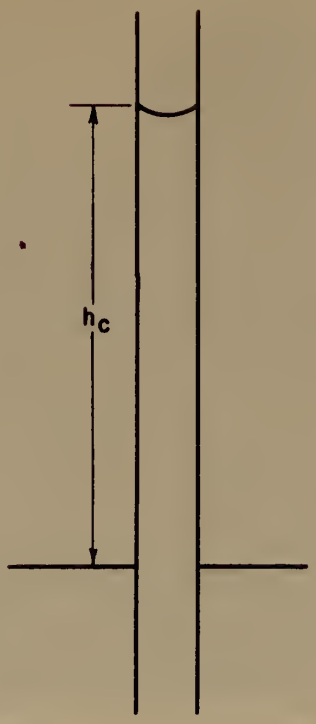



Figure V B-10.



Figure V B-11.

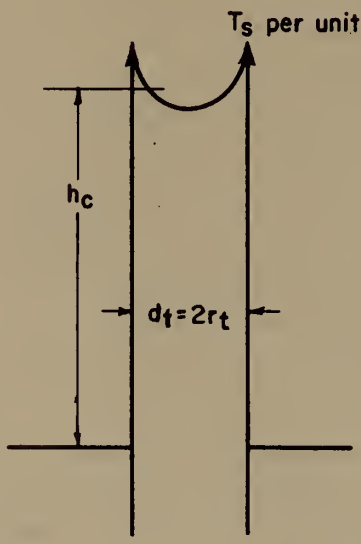



Figure V B-12.



Figure V B-13.



Figure V B-14, a - DUKW TRACKS IN WET PACKED SAND AT LOW TIDE.



Figure V B-14, b - DUKW TRACKS IN MOIST WIND-BLOWN SAND.

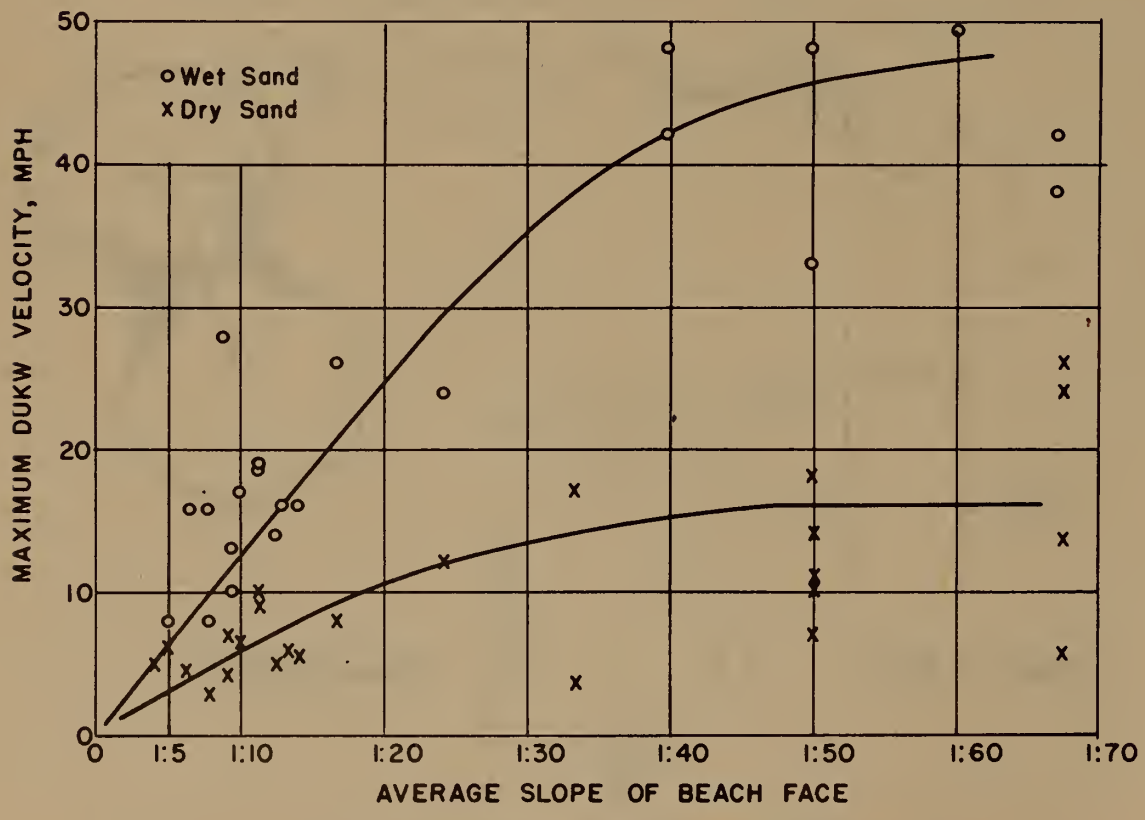



Figure V B-15 - MAXIMUM DUKW VELOCITY RELATED TO BEACH FACE SLOPE



a. Jeep stuck on dry berm, parallel to beach, heading up cusp.



b. Dukw stuck in hole made by an LCVP during landing.



c. Tractor takes three tries to negotiate a scarp.



d. Tractor throws track in attempt to go up a scarp at an angle.

FIGURE V B-16 – EFFECTS OF CUSPS, SCARPS AND DEPRESSIONS ON TRAFFICABILITY



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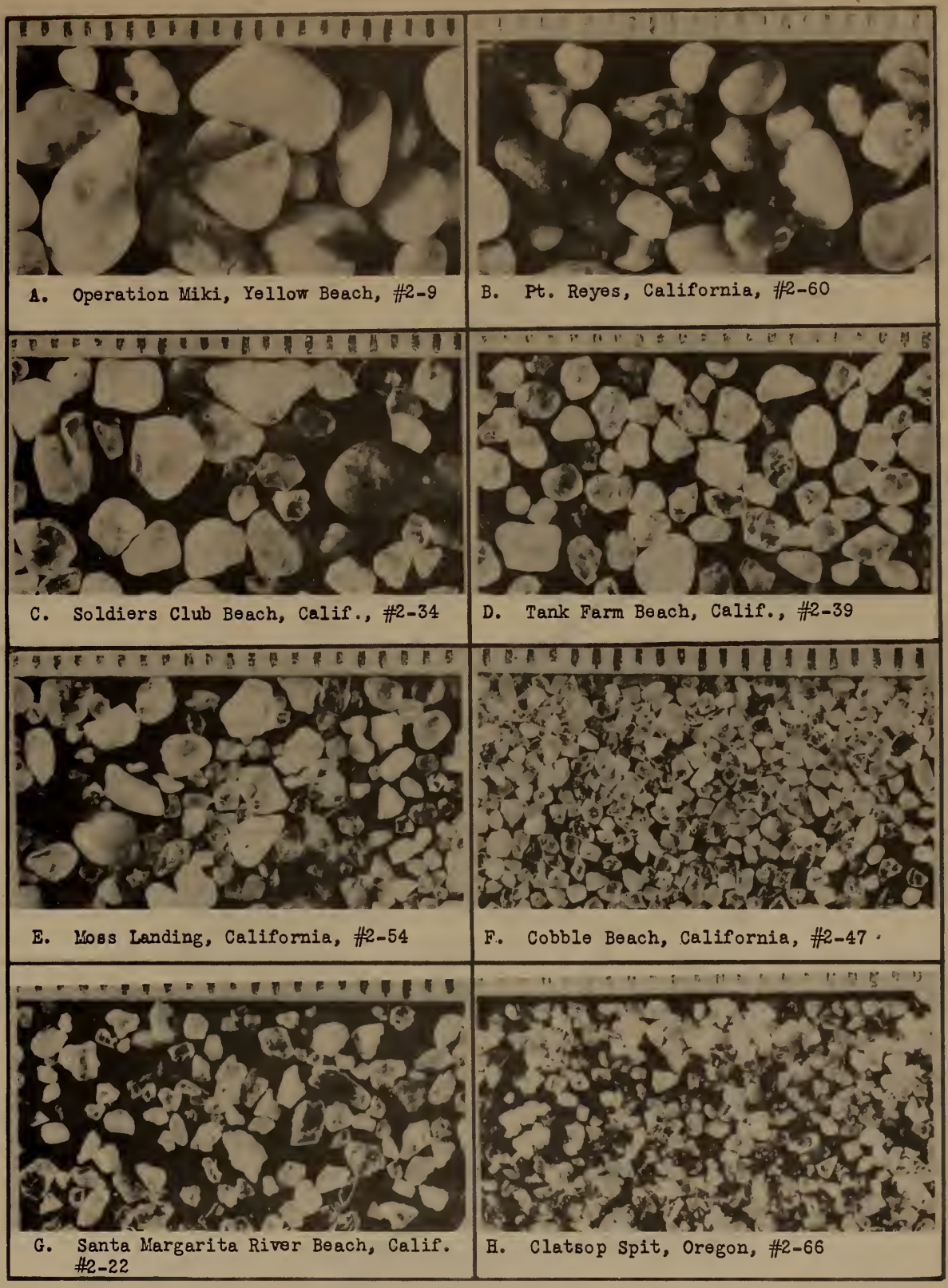


a. MONTEREY



b. CLATSOP SPIT

FIGURE V B-17 - EFFECT OF BACKSHORE CONDITIONS ON TRAFFICABILITY



A. Operation Miki, Yellow Beach, #2-9

B. Pt. Reyes, California, #2-60

C. Soldiers Club Beach, Calif., #2-34

D. Tank Farm Beach, Calif., #2-39

E. Moss Landing, California, #2-54

F. Cobble Beach, California, #2-47

G. Santa Margarita River Beach, Calif. #2-22

H. Clatsop Spit, Oregon, #2-66

Fig. V B-18 Comparison of Photomicrographs of Sand Samples Arranged From Steepest Beach to Flattest Beach Studied. 1/64 Inch Between Vertical Marks.

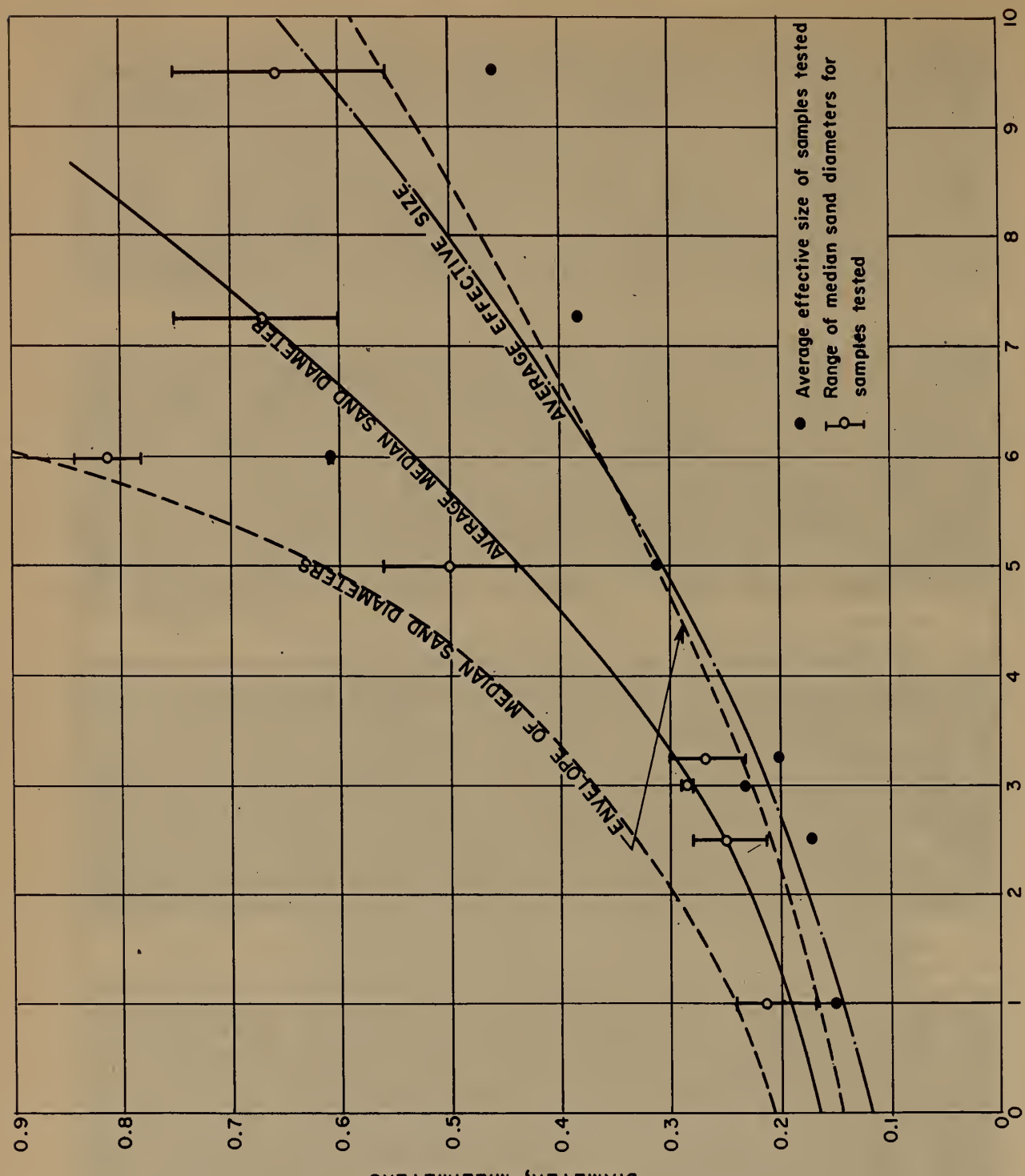



FIGURE VB-19 - CORRELATION OF MEDIAN SAND DIAMETER & EFFECTIVE SIZE WITH BEACH FACE SLOPE

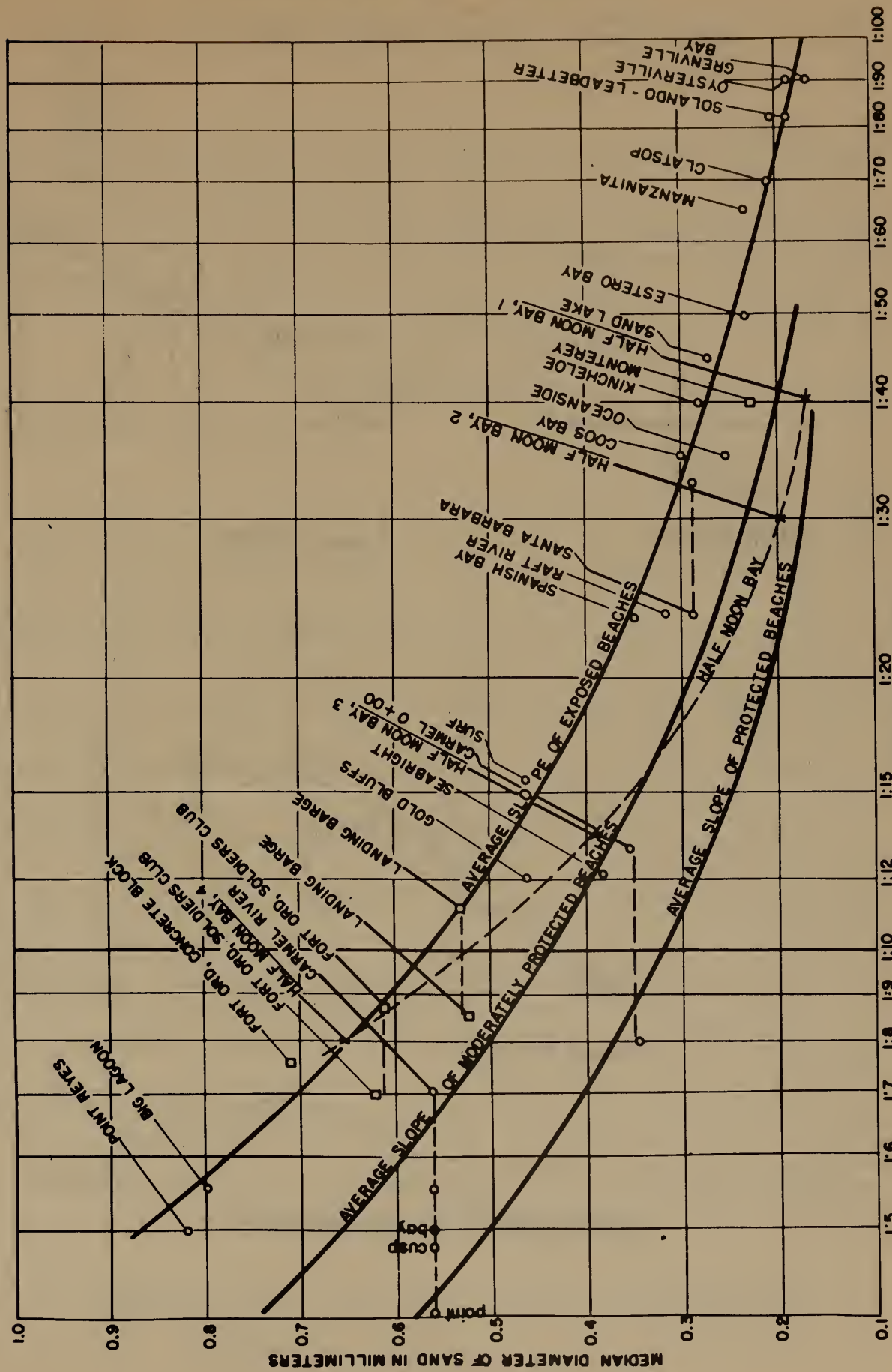



FIGURE VB-20 - SIZE-SLOPE RELATIONSHIP AT THE REFERENCE POINT

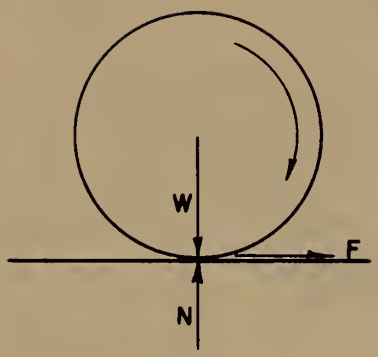



Figure V B-21.

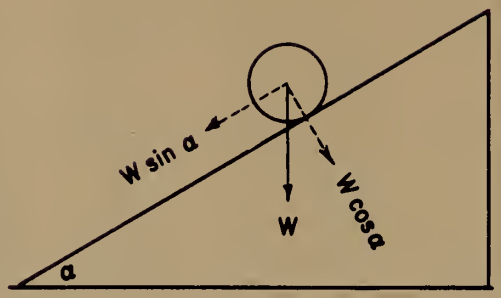



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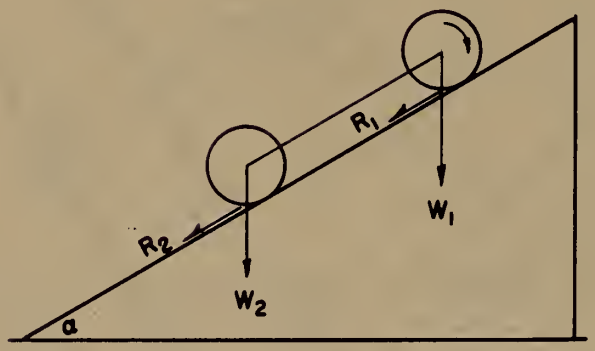
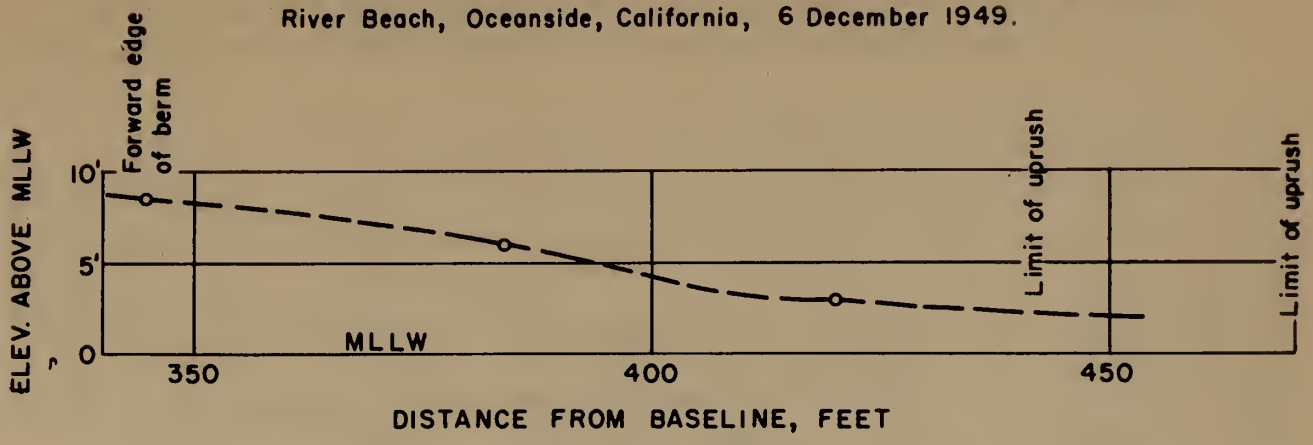



Figure V B-23.

Cone Penetrometer measurements along Range 7+50, Santa Margarita River Beach, Oceanside, California, 6 December 1949.




BEACH PROFILE

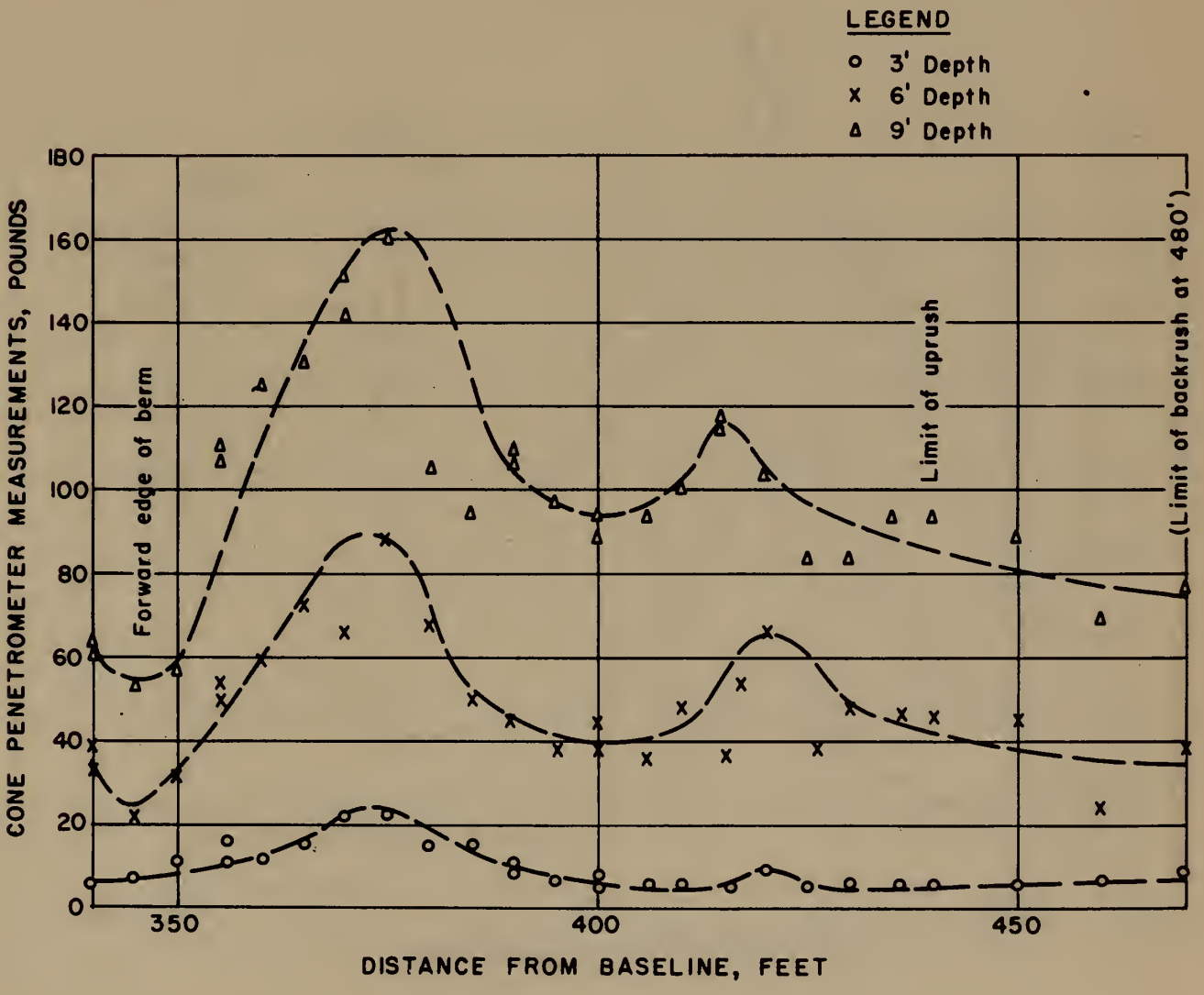
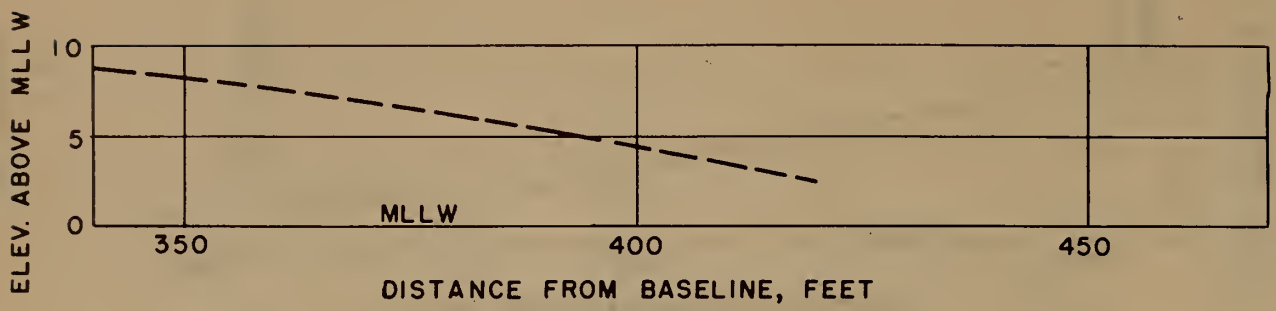



FIGURE VB-24 - CONE PENETROMETER MEASUREMENTS

Cone Penetrometer readings and beach face profile, Range 7+50 N,
Santa Margarita River Beach, Oceanside, California, 29 November 1950.




BEACH PROFILE

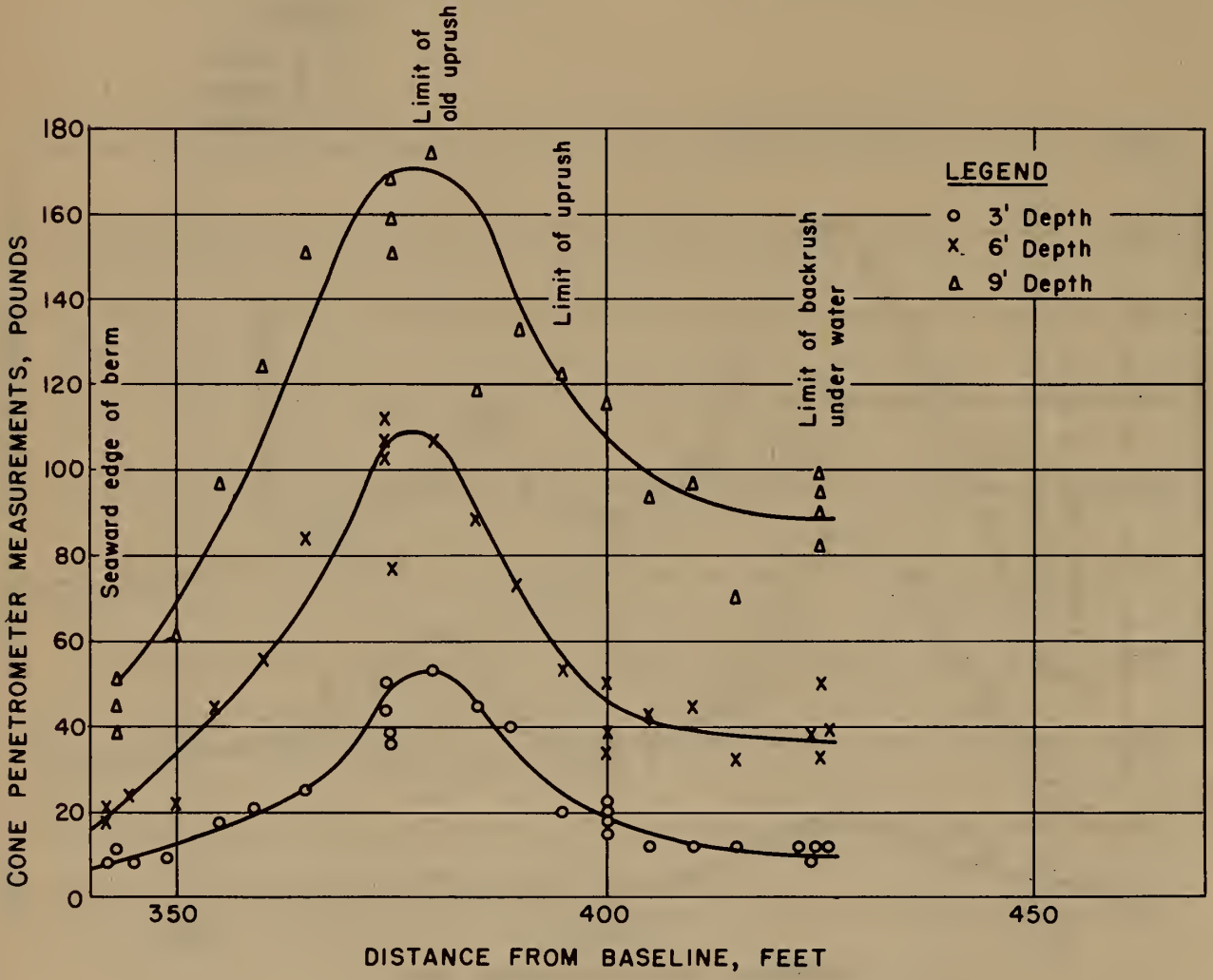
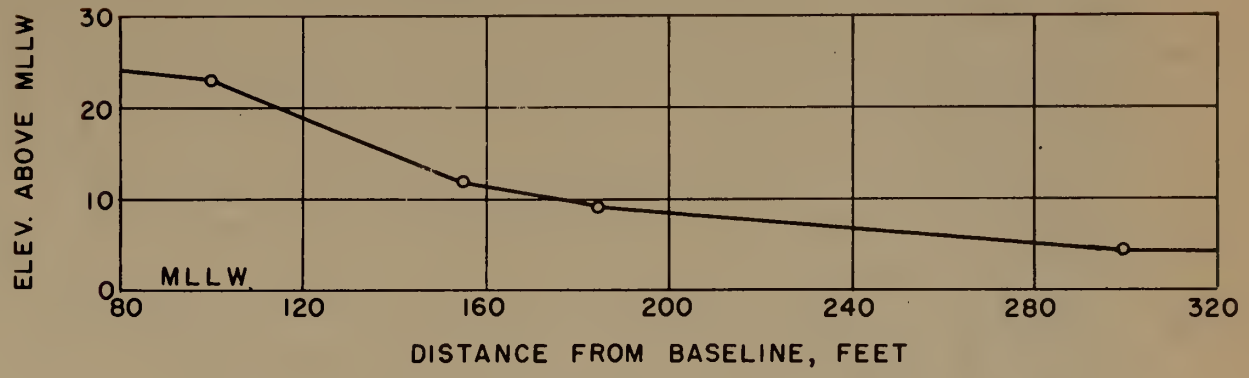



FIGURE V B-25 - CONE PENETROMETER MEASUREMENTS

Cone Penetrometer measurements and beach profile, Range 0+00, Test Site, Clatsop Spit, Oregon, 28 April 1950.




BEACH PROFILE

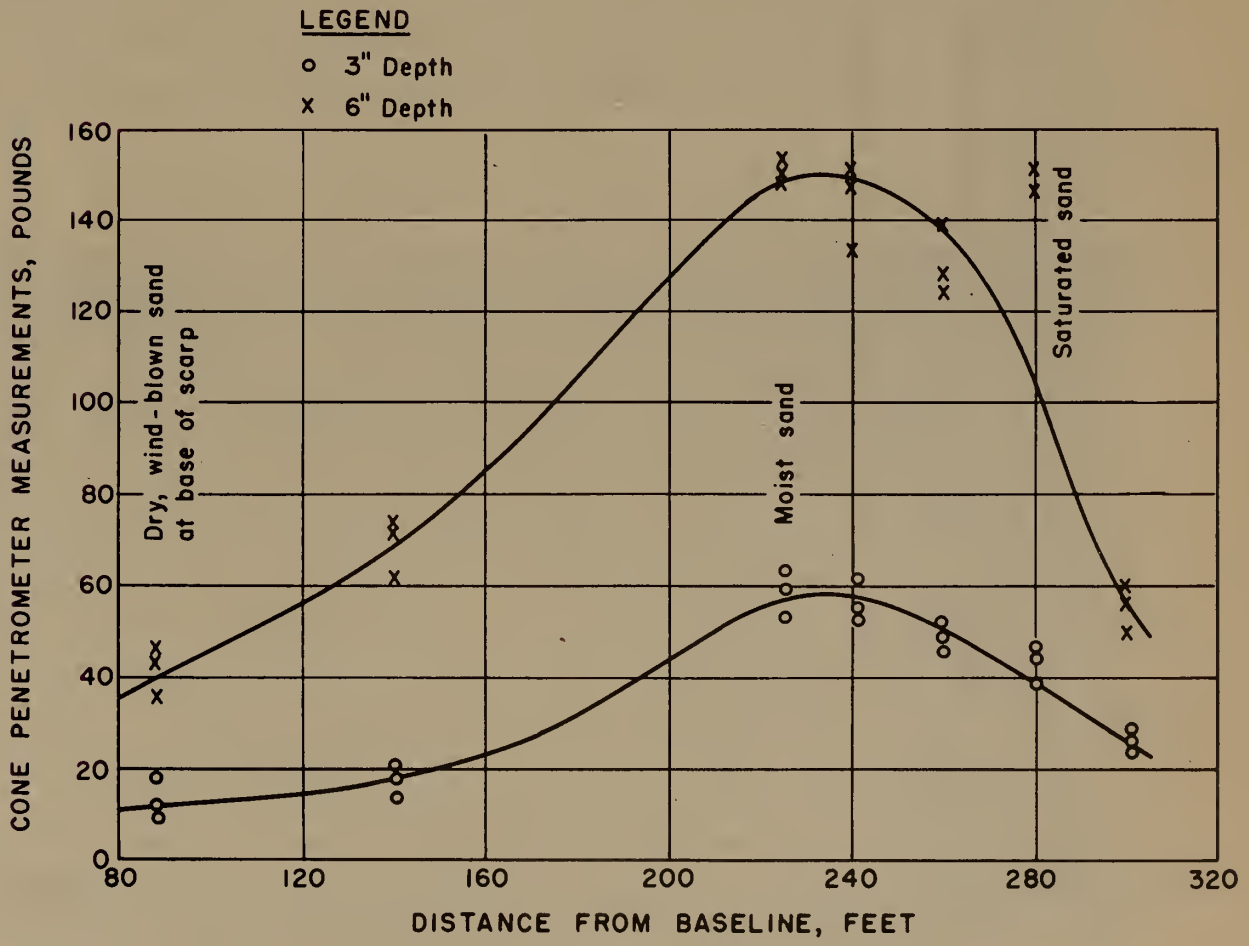
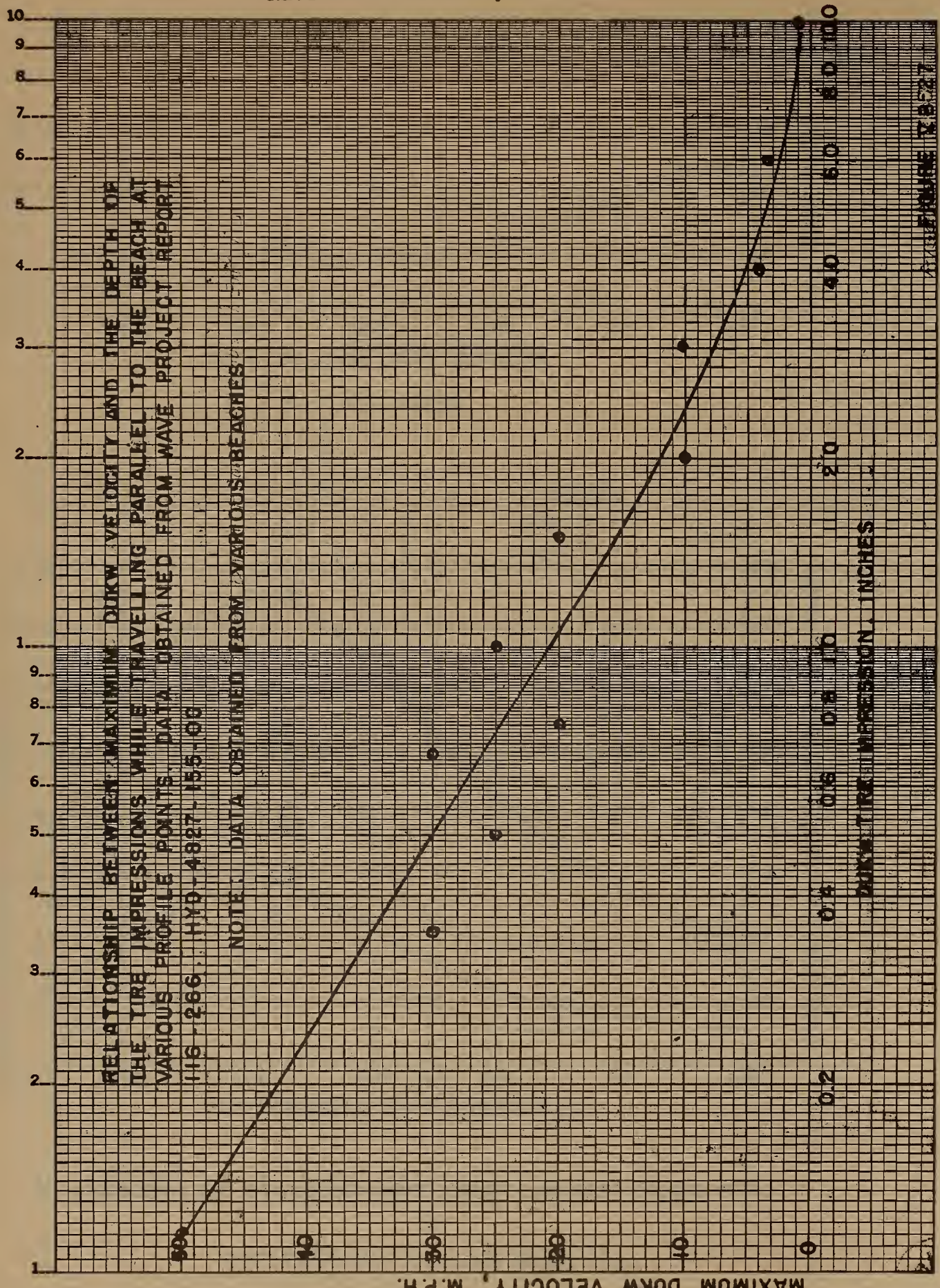



FIGURE V B-26 - CONE PENETROMETER MEASUREMENTS


RELATIONSHIP BETWEEN MAXIMUM DUKW VELOCITY AND THE DEPTH OF THE TIRE IMPRESSIONS WHILE TRAVELLING PARALLEL TO THE BEACH AT VARIOUS PROFILE POINTS. DATA OBTAINED FROM WAVE PROJECT REPORT NO. 266, HYD-4827-15-06
NOTE: DATA OBTAINED FROM VARIOUS BEACHES
FIGURE 13-27
MAXIMUM DUKW VELOCITY, M.P.H.
DUKW TIRE IMPRESSION, INCHES
0
10
20
30
40
0.2
0.4
0.6
0.8
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10

C. PREDICTION OF TRAFFICABILITY

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   b. Rolling Resistance - - - - - - - - - - - - - - - - - - 3

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## C. PREDICTION OF TRAFFICABILITY

### 1. Rational Approach to Prediction of Trafficability

An attempt to predict the trafficability of a soil by a rational consideration and determination of some of the fundamental factors involved has been made by R. C. Stewart and S. J. Weiss of the U. S. Naval Civil Engineering Research and Evaluation Laboratory, Port Hueneme, California (VB-9). The following analysis of the drawbar pull developed by a vehicle is taken from their report.

"The index of performance of a prime mover is the amount of load it can pull in addition to moving itself. Thus the prediction of performance of a tractor is the estimating of the drawbar pull available for pulling a load.

"The index of performance of a self-propelled vehicle not used as a prime-move, is merely whether or not passage of terrain is possible. This corresponds to a prime-mover being able to move itself but not being required to pull a load. This, if the vehicle is analyzed in the same manner as a tractor and the predicted drawbar pull is anything above zero, capacity to pass over the terrain is indicated.

"The index of performance of a towed vehicle is the load a prime-mover would have to exert as drawbar pull in order to move the vehicle. This load is the sum of the internal and external rolling resistance of the towed vehicle.

"The prediction of vehicle performance thus involves:

(i) For a prime-mover or self-propelled vehicle, the estimating of drawbar pull

(ii) For a towed vehicle, the estimating of total resistance.

The drawbar pull of a prime-mover on level ground can be predicted from the following analysis. The law of conservation of energy requires that:

| Power supplied to sprocket of track of vehicle | = | Power expended in overcoming friction force when slippage occurs | + | Power expended in pulling load | + | Power expended in moving vehicle against rolling resistance |
|---|---|---|---|---|---|---|

$$550H = F(1 - k)v + P.k.v + Rkv$$

where H = available drawbar horsepower

F = tractive force

P = drawbar pull

R = rolling resistance = external rolling resistance + internal rolling resistance

k = slip factor; k = 1 (no slippage); k = 0 (100% slippage)

peripheral velocity of wheel or track.

Also, the limitations imposed by the soil are

$$F = P + R$$

since the sum of all horizontal forces must be zero if the vehicle moves at constant velocity.

From above:

$$\frac{550H}{V} = F(1 - k) + Pk + Rk$$

$$= F(1 - k) + Fk$$

$$= F$$

Hence, the maximum tractive force as determined by the power of the vehicle is $\frac{550H}{V}$ and the maximum drawbar pull is therefore

$$P = F - R = \frac{550H}{V} - R$$

However, the value of F is also limited by the traction of the wheel or track on the soil. If the maximum value of F limited by traction is $F_t$, the maximum drawbar pull is

$$P = F_t - R$$

The actual limiting condition of operation will be determined by the lesser of the two possible values of P."

Hence, the problem is resolved into the determination of the tractive force and the rolling resistance. The maximum drawbar pull as limited by power is a mechanical feature of the vehicle and may be obtained from the vehicle specifications. A method of evaluating the maximum tractive force as limited by traction is also available but the rolling resistance cannot be readily determined. Proposed methods for determining the tractive force and the rolling resistance are discussed in the following pages.

a. Tractive Force

The work of the Canadian Directorate of Vehicles and the Aberdeen Proving Ground has resulted in the following formula for the tractive force developed by a conventional closed-link track on soil:

$$F = V\left[(1 + F_h)\tan\phi + F_h F_s \tan^2\phi\right] + 2cL(w + 2h)$$

where

F = tractive force

$$F_h = 0.64\,\frac{h}{w}\cot^{-1}\frac{h}{w}$$

$$F_s = 0.64\,\frac{s_g}{w}\cot^{-1}\frac{s_g}{w}$$

h = height of grouser

w = width of track

$s_g$ = spacing of grousers

c = effective cohesion

$\phi$ = effective angle of friction

This equation has been closely corroborated by experiment.

Further information is desirable on the tractive force which may be developed by rubber-tired vehicles, with and without traction devices, on various types of soils.

b. Rolling Resistance

A method for determining the rolling resistance of a wheel or track has been proposed by Stewart and Weiss. This analysis will be presented in outline form since it indicates a method of attacking the problem of estimating rolling resistance.

The authors state that the analysis is based on the following assumptions:

(i) The intensity of ground pressure $p_v$ on a rolling wheel is constant for the full area of ground contact.

(ii) The intensity of the resistance pressure $p_r$ is constant for the full frontal area of ground resistance.

(iii) The soil under the wheel acts as an elastic material.

The latter assumption is somewhat misleading since the elastic properties of the soil do not appear in the analysis and elastic theory is used only for the purpose of determining the maximum shear stress in an element of soil subject to triaxial stresses. This shear stress and the normal stress on the same plane are then substituted in an equation defining the condition of plastic failure of the soil element, whereas in fact, failure does not occur on the plane of maximum shear stress. The error introduced in the analysis due to this cause is small but the manner in which the assumption of an elastic soil is used in this case may be considered to detract rather than add to the accuracy of the analysis.

Based on the above assumptions the forces acting on a rolling wheel are analyzed as follows. A wheel rolling at constant speed is in static equilibrium in that the sum of the external moments is zero. (If the sum of the moments is not zero the wheel will slip insteal of roll.)

Hence, using the symbols shown on Figure VC-1 where R is the resistance to rolling caused by the soil ahead of the wheel:

$$\frac{R'(D-z)}{2} = \frac{V\sqrt{Dz-z^2}}{2}$$

But unit vertical pressure = $p_v = \frac{V}{w\sqrt{Dz-z^2}}$

and unit resistance pressure = $p_r = \frac{R'}{wz}$

$$\therefore p_r \, wz \cdot \frac{D-z}{2} = p_v \cdot w \cdot \frac{Dz-z^2}{2}$$

$$\therefore p_r = p_v$$

By an analysis very similar to that first used by Rankine for the determination of bearing capacity, the relationships between the vertical and horizontal stresses $p_v$, $p_z$ and $\gamma z$ acting on elements of soil immediately below and adjacent to the bottom of the wheel are then found. The relationship between $\gamma z$ and $p_z$ is established by drawing the Mohr circle for the stresses acting on the soil element adjacent to the bottom of the wheel and determining the condition of plastic equilibrium of the element assuming the shear strength is denoted by

$$s = c + p \tan \phi.$$

For this purpose the Mohr circle for the element is considered to be tangent to the envelope of failure defined by the shear strength formula. The relationship between $p_z$ and $p_v$ is found in a similar manner except that the Mohr circle for the element immediately below the wheel is drawn so that the envelope of failure passes through the point of maximum shear stress in the element, as shown in Figure VC-2. In addition, an impact factor, m, is introduced since impact will cause the effective vertical pressure to be somewhat greater than $p_v$.

Hence, the following result is obtained:

$$p_v = \frac{\gamma z N_\phi + 2c\sqrt{N_\phi} + \frac{2c}{1 + \tan \phi}}{\frac{m\,(1 - \tan \phi)}{1 + \tan \phi}}$$

where

$$N_\phi = \tan^2 (45 + \frac{\phi}{2})$$

But it has already been shown that

$$p_v = \frac{V}{w\sqrt{dz-z^2}}$$

The two values of $p_v$ can be equated and the resulting equation solved for the sinkage z since the other factors V, w, D, $\gamma$, c and o can be determined. When z is known, the rolling resistance R can be computed from equilibrium considerations of the wheel.

The main criticisms of this analysis, in addition to those already mentioned, may be summarized as follows:

(i) The intensity of ground pressure and the intensity of resistance

pressure may be approximately uniform for wheels rolling on cohesive soils but they will not be uniform for cohesionless soils.

(ii) The same analysis could be applied to elements at any depth between zero and z, the full depth of sinkage.

(iii) The relationship between $p_z$ and $p_v$ is determined by substituting in a failure relationship, values of shear stress and normal stress for a plane on which failure does not occur.

(iv) The value of $p_v$ when m = 1 represents the bearing capacity of the soil for static loads.

(a) when $\phi = 0$, $N_\phi = 1$ and $p_v = \gamma z + 4c$.

For the type of remoulded clays encountered in trafficability problems this value is somewhat less than the value of $\gamma z + 5.5c$ obtained by other analyses and verified by experimental results.

(b) when $c = 0$, $p_v = \gamma z \cdot \frac{1 + \sin \phi}{1 - \sin \phi} \cdot \frac{1 + \tan \phi}{1 - \tan \phi}$

This value of the bearing capacity of a cohesionless soil has been shown to be very much lower than the actual bearing capacity by numerous investigators.

(v) In view of the conservative values of $p_v$ given by the formulas, reasonable values for z can only be obtained by selecting convenient values of the factor m which are not necessarily related to impact effects.

(vi) The analysis shows that the surface bearing capacity of a cohesionless soil is zero; this is not so.

(vii) The bearing capacity of a soil for which the effective angle of friction is greater than zero is known to vary with the width of the loaded area. This is not shown by the analysis.

(viii) The authors suggest the use of an impact factor m = 2 for wheeled vehicles. When vehicles can go at relatively high speeds high impact effects may occur but trafficability will not be a problem. When trafficability is a problem vehicles have low speeds and the soil is relatively soft, both of which will tend to reduce the impact factor considerably below 2.

(ix) The analysis takes no account of the increase in strength of soils under transient loads. This effect may be of sufficient magnitude to neutralize any impact effect.

(x) The compressibility of the soil is neglected.

(xi) Additional factors which contribute to the rolling resistance, such as stickiness, are neglected.

The authors recognize some of these shortcomings of the theory and the need for refinement of the analysis. In spite of its limitations, however, the analysis is of interest because it represents the first attempt to determine trafficability in terms of fundamental soil properties. In this case the unit weight and strength characteristics only are required. In addition, the compressibility of the soil under transient loads, the stickiness and the slipperiness would be required for a more complete analysis.

The analysis also indicates the method of attack once these soil properties have been determined. A satisfactory formula for determining the tractive force developed by a tracked vehicle is available. A similar formula is required for the tractive force developed by wheeled vehicles both with and without traction devices. In addition, a more refined theory is required to evaluate the rolling resistance. From these results the drawbar pull developed by the vehicle, which is the index of trafficability, could be obtained.

c. Measurement of Strength Characteristics - the Soil Truss

The most important soil properties determining trafficability are the shear strength characteristics - the effective cohesion and the effective angle of friction. As a further contribution to the determination of trafficability, Stewart and Weiss have also developed an instrument for measuring these characteristics of the soil in the field. This instrument is called the soil truss.

The authors describe the soil truss as the field counterpart of the direct shear box for measuring soil strength. "It consists of two legs: (i) an active leg to which is attached an active shoe, the function of which is identical with that of the upper frame of the direct shear box; and (ii) an anchor leg, to which is attached an anchor shoe to provide stability and support for the instrument. The ground itself is the counterpart of the lower frame of the direct shear box. The anchor leg is designed to remain fixed in the ground while the active shoe shears its entrapped prism of soil. The soil is cleared from the front and sides of the active shoe in order to eliminate lateral flow effects."

A photograph of the soil truss is shown on Figure VC-3 and detail drawings are presented in Figure VC-5. The active shoe consists of a separate anvil and frame in order that all of the applied load will be transferred to the test specimen of soil. The specimen has a circular cross-section in order to minimize the error caused by variation of cross-sectional area when the specimen is sheared. Load is applied through a standard Proctor Needle barrel and measured by calibration of the barrel. Knowing the leg angle, the normal and shear stresses on the specimen at failure can be determined.

Operation of the truss at various leg angles enables the shear strength of the soil under various normal stresses to be determined. These results can then be plotted to determine the effective cohesion and the effective angle of friction for these particular loading conditions. A typical set of test results obtained with this instrument are shown on Figure VC-4.

2. Empirical Approach to the Prediction of Trafficability

The ability of a soil to provide adequate bearing capacity for a given vehicle and the tractive force developed by grousers or treads which dig into the top layer of soil depend mainly on the shear strength of the soil. If the bearing capacity is low the vehicle settles and it is necessary for the vehicle

push soil out of the way in order to move. This is usually the major cause of rolling resistance. If the layer through which the vehicle settles is soft it can be pushed away easily but if it is stiff it adds considerably to the rolling resistance. The increase in effort required to push away soil as the vehicle progresses is also a function of the shear strength of the soil.

In addition to shear strength, slipperiness and stickiness also affect vehicle performance but shear strength is the most important single factor in determining trafficability. Owing to the difficulty of analyzing the stresses in a soil mass under a wheel load and thus determining the sinkage and rolling resistance, various attempts have been made to predict trafficability by an empirical correlation of vehicle performance with the shear strength of the soil. In these correlations the shear strength has not been measured directly but some indicator test has been used which is itself related to shear strength. In order to simplify the procedure as much as possible the indicator test has usually taken the form of some type of penetration device. Two main types of penetration device have been used--cone penetrometers and dynamic penetration devices.

a. Cone Penetrometers

In these instruments the resistance offered by the soil to the slow penetration of a cone is measured. The original North Dakota Cone apparatus (VC-2) is shown in Figure VC-6a. In this test the cone, which has an apex angle of 15° 30', is placed so that the tip is in contact with the soil. A load is then placed on the platform sufficient to give a 10 lb. load on the cone and a set screw is released, allowing the load to be applied slowly. When movement has stopped the depth of penetration is noted and the process is then repeated for loads of 20, 40 and 80 lbs. The penetration of the cone into the soil is used to compute the projected area of the cone buried in the soil, which is divided into the load to give the cone bearing value in lb. per sq. inch. Generally the values are reasonably constant for the different loads though tests by the U. S. Corps of Engineers (VB-14) have shown that this does not apply in all soils especially sands. Furthermore, the apparatus is relatively heavy and the instrument has therefore been modified in order to obtain a light and more portable penetrometer for field use.

Cone penetrometers were developed independently by the U. S. Navy and the Corps of Engineers though the instruments are basically alike and differ only in minor details. Each consists of a cone, having a 30° angle at the apex, to which load may be applied through a rod and calibrated spring or proving ring. The cone is pushed into the soil to the depth at which a reading is desired. The load is then reduced slightly and then gradually increased and the pressure at which movement just starts is recorded.

The Corps of Engineers cone penetrometer (VB-13 and 14), which has been used in numerous trafficability studies is shown in Figure VC-6B. Details of this instrument are shown in Figure VC-7. It consists of a 30° cone with a 1 sq. in. projected area mounted on a 5/8" diameter shaft. Load is registered by a proving ring and the point at which movement begins is determined by "feel". The load in pounds at which movement first occurs is called the cone index of the soil.

The following note on the use of this penetrometer is extracted from a report from the Waterways Experiment Station, Vicksburg (VB-13). "The operation of the cone penetrometer was very simple under practically all circumstances.

No difficulty was experienced in obtaining readings in the test lanes in either dry or inundated areas and readings could be taken at intervals of depth without removing the penetrometer. Reasonably consistent results could be obtained by different operators. The difficulty experienced in determining just when the cone starts moving, which was inherent in the spring type instrument, was largely eliminated by substituting the proving ring as a load measuring device for the spring used in the 1945 instrument. The movement in the spring-type instrument was approximately 3 inches and was so great that it was impossible to determine movement without direct contact with the cone. In the new model the movement in the load registering device is negligible. Movement of the cone can be determined by feel without direct contact with the cone. Learning the 'feel' does not require an extensive training period. The feel is difficult to describe but on several occasions visiting engineers were asked to try the instrument. After one or two trials they were able to determine with ease when movement of the cone started (BV-15)."

Another type of cone penetrometer is the B.T.U. Beach Tester, Model 2, developed by the Beach Trafficability Unit of the Joint Army-Navy Experimental and Testing Board (VC-3). This device, shown in Figures VC-6C and VC-8 consists of a 2 inch diameter cylinder approximately 24 inches long in which a coil spring, resting on a piston, is mounted. A rod into which various sizes of cone or disc points can be screwed is attached to the piston and extends through the length of the cylinder and below it an additional 14 inches. A grip handle is attached to the upper end and two foot rests are attached to the lower end of the cylinder. The upper one foot of the rod is graduated in inches.

The instrument is operated by pushing the point into soil until the two foot rests are flush with the ground. The penetration of the point in inches is then read from the graduations on the upper end of the rod. This device was designed in an attempt to provide an instrument that would give a very quick indication of the hardness of beach materials. Very few tests have been made with this instrument and there has been no correlation of the results with trafficability.

b. Drop Cone and Dynamic Penetration Devices

In this type of penetration device a moving cone strikes the surface of the soil and the dynamic resistance of the soil is measured by the depth of penetration of the cone.

The earliest device of this type is the British "Campbell Stick" (VC-3) shown in Figure VC-6d. A plunger fitted with a detachable foot is projected into the soil by means of a compressed spring. The foot, 3/4" in diameter, may be either flat or tapered. The flat foot was designed for use in sand and the tapered foot for use in Clay. In operation the plunger is raised to compress the spring and then held in position by means of the pawl. The base plate is held firmly against the ground surface by means of the foot rest and the plunger released. A pointer attached to the plunger shows the depth of penetration. A British report (VC-1) describes the relationship between readings of this instrument and the trafficability of Sherman and Churchill tanks.

A modified form of the Campbell stick was developed in the United States by the JANET Beach Trafficability Unit and called the B.T.U. Beach Tester, Model 1. This instrument is shown in Figures VC-6e and VC-9. It is described in the Beach Trafficability Unit report (VC-3) as follows: "The instrument has

a slotted Tubular steel casing in which is housed a coiled spring. The spring seats against the top of the casing and the lower end presses against a steel plunger. To the latter is screwed one of the various types of operating feet or bearing points. Attached to the plunger are two connecting pins which engage the cocking mechanism at full spring compression. Twelve raised 1-inch graduations are provided on the outside of the casing adjacent to one of the slots. The penetration is read by noting the length of travel of the plunger pins. A handle at the top of the casing and two foot rests (stirrups) attached to the bottom are provided. Various types of operating feet are screwed into the end of the plunger. When cocked the penetrating plunger extends 3 inches below the bottom of the instrument. In very soft material this allows the tip of the plunger to penetrate a maximum distance of 15 inches below the ground surface. The spring develops a force of 80 lb. under maximum compression.

"The instrument is cocked by pushing the plunger against a firm footing until the projecting pins engage the trip. The projecting part of the plunger is pushed into the soil, the operator's feet are placed firmly on the footrests and the trip is released. The depth of penetration of the plunger is then read from the raised scale. This value as recorded by the operator does not include the 3 inches the plunger penetrates before the spring is released. The graduations for reading the depth of penetration are raised so that readings may be taken by day, by night or under water by sense of touch. The instrument has to be cocked for each reading and the projecting pins will only engage the trip when the spring is at maximum compression. This eliminates a source of error that would occur if the operator did not fully cock the spring. Although the instrument is designed to eliminate human error as much as possible, variations in the penetration of the foot occur if the top of the device is moved during operation, especially in soft silty sands".

Tests were carried out with this instrument on various beaches where the soils ranged from medium sand to fine silty sands. These tests indicated that it would be necessary to use more than one size of foot in order to cover the trafficability limits of all wheeled and tracked vehicles but insufficient tests were conducted to establish a correlation between the instrument readings and vehicle performance.

The Waterways Experiment Station, Vicksburg (VB-13) also developed a drop-cone instrument which had the apparent advantage of eliminating the human error which is inherent to some extent in the cone penetrometer. The drop-cone is shown in Figure VC-10. It consists of a 30° cone having a base area of 1 sq. in. attached to a plunger of the same cross-section area. The plunger and cone weigh 1.25 lb. The cone is held with its tip 4 inches above the ground surface and allowed to fall, the depth of penetration being determined by a scale on the side of the plunger. Operation is simple and rapid except when water is present. In this case it is necessary to set a short section of casing and bail the water before the test can be made. Again, when readings are required below the ground surface difficulty is encountered in boring or excavating a hole with a sufficiently smooth bottom. The need for casing when water is present and the difficulty of taking readings below the ground surface are serious drawbacks of the drop-cone. Furthermore, readings of the drop-cone were not so consistent as those of the cone penetrometer. Consequently the use of the drop-cone was discontinued in favor of the more easily used cone penetrometer.

c. Use of Cone Penetrometers in Trafficability Studies

Although various types of penetration devices for predicting traf-

ficability have been developed, the U. S. Corps of Engineers' cone penetrometer, Figure VC-6b has been most widely used in trafficability studies. The reading obtained from this instrument is called the cone index. Extensive series of tests have been performed by the University of California (VB-15) in an attempt to correlate the cone index with the trafficabilities of sandy beaches. The Waterways Experiment Station, Vicksburg, has also published several reports (VB-12, 13 and 14) on the correlation of cone index with the results obtained from other tests used to measure bearing capacity and with the trafficability of cohesive soils. The following results are obtained mainly from these reports.

d. Variation of Cone Index With Water Content And Density

Tests were conducted at the Waterways Experiment Station to determine the cone index and dry density of various types of soils compacted at varying water contents using three different compactive efforts. The compactive efforts were chosen to give specimens with dry densities approximating those of field conditions and specimens were usually prepared at 6 to 10 different water contents ranging from wet to dry.

For a given compactive effort and over the range of water contents investigated, the cone index varied with water content as follows:

(i) For sands and gravels, maximum cone index occurred at some optimum water content. For water contents above and below the optimum the cone index decreased.

(ii) For sandy silts there was some indication of an optimum water content although tests were not conducted at sufficiently low water contents to establish an optimum.

(iii) For clays and clayey silts the cone index decreased with increasing water content.

In all cases there was an optimum water content when dry density was plotted against water content. The optimum water content for cone index was usually slightly lower than the optimum water content for dry density.

Different compactive efforts produced different dry densities for specimens compacted on the dry side of optimum but as the water content increased above optimum, different compactive efforts resulted in about the same dry densities; where increased compactive effort produced greater densities it also tended to produce greater values of cone index though this was not always the case. In many cases, for the fine grained soils, the cone index of samples of the same soil at the same water content was the same even though the samples had different densities.

In general, the relationship between dry density, cone index and water content had the form shown in Figure VC-11. At a certain optimum water content, $W_{co}$, the cone index reached a maximum for the different compactive efforts. At a water content $W_{cc}$ which was slightly greater than $W_{co}$ the cone index became independent of the compactive effort; that is, different compactive efforts produced the same cone index. Approximately $W_{cc}$ was equal to $W_{do}$, the optimum water content for maximum dry density for the different compactive efforts. At the water content $W_{cc}$ and at water contents just above it, dif-

ferent compactive efforts produced different dry densities though they produced the same cone index. At a water content a few percent greater than $W_{cc}$ another water content $W_{dc}$ was reached above which the dry density was independent of the compactive effort. For water contents greater than $W_{dc}$ different amounts of compaction produced both the same cone index and the same dry density.

The data obtained from tests on sands and gravels were unsufficiently conclusive to allow any conclusions to be drawn. For example, for Fort Pierce sand none of compactive efforts at any water content produced a cone index greater than 20. For the gravel or gravelly sand, a cone index greater or less than 50 could be obtained depending on the amount of compaction and the water content.

For the plastic soils, the cone index decreased with increase in water content for the wide range of water contents investigated. The cone index at the water content $W_{cc}$ ranged from 30 to 80 for the different silts and clays with an average of about 50. At this water content the dry density of the samples varied as much as 6.5 lb/cu.ft. with an average of about 3 lb./cu. ft. with no corresponding change in cone index. It is thus seen that for this particular cone index, and for water contents just above it (between $W_{cc}$ and $W_{dc}$) density was not a major factor in determining cone index. The results also indicated that this conclusion holds for all water contents greater than $W_{cc}$, although when different compactive efforts produce the same dry density and the same cone index, both of which vary with water content, a definite relationship must exist between cone index and dry density. However, as is indicated by the data, this relationship is not necessarily of fundamental importance. These results, expressed in terms of cone index, indicate that for cone index values less than about 50, the density of the soil is not a major factor in determining the cone index. This range of cone index values is very important from the standpoint of trafficability since it is the range which produces the most serious problems.

No simple relationship between water content, dry density and cone index could be determined at the lower water contents where density was apparently a factor. Each soil showed different characteristics. In general, there was a definite increase in cone index with an increase in density. Specimens compacted 60 blows per layer, which produced the highest densities, showed 50 to 200 percent greater cone index values than those compacted 20 blows per layer at the water content which produced maximum compaction. However, the cone index in these cases was usually above 50, which for silts and clays, represents a material that will support most vehicles. From this standpoint, the range of conditions where density was a factor in determining cone index was of less interest than the range where it was not.

In the plastic soils there was also found to be a tendency for the cone index to increase with depth but in general the increase was insignificant.

e. Correlation of Cone Index and California Bearing Ratio

The California Bearing Ratio (CBR) is an index of the bearing capacity of a soil which is used in the design of flexible pavements. The correlation of cone index with CBR has been found to be reasonably consistent for fine grained plastic soils but no consistent correlation has been obtained in tests on granular soils. For example, at a CBR of 5, the following variation of cone index with soil type has been determined:

Table VC-1

Variation of Cone Index of Granular Soils at Constant CBR

| Soil | Cone Index | CBR |
|---|---|---|
| Shreveport Gravel | 70 | 5 |
| Shreveport Gravelly Sand | 45 | 5 |
| Fort Pierce Sand | 8-20 | 5 |
| Fort Pierce Silty Sand | 50 | 5 |
| Shreveport Clayey Sandy Silt | 160 | 5 |

In the test on gravels and sands at very low water content the CBR showed fairly high values whereas the cone index was low. This is probably due to the fact that the surcharge used in the CBR test tends to confine material whereas no surcharge is used in the determination of the cone index at the soil surface. For a surface reading on these materials the CBR test is probably more reliable since the tracks and tires of vehicles provide some confining effect.

For plastic soils and for most non-plastic soils it has been found possible to establish an approximate relationship between cone index and CBR. For about 12 plastic soils tested, ranging from clayey silts to highly plastic clays, there was found to be some general agreement between the cone index-CBR relationships. For these soils the cone index was roughly 30 times greater than the CBR. Thus a CBR of 3 corresponds to a cone index of about 90. The range of cone index values from 0 to 90 measures the extreme low range of strengths on a magnified scale as compared with CBR values.

f. Correlation of Cone Index and North Dakota Cone Bearing Value

Reasonably well-defined relationships between cone index and North Dakota Cone bearing values have been found for a wide variety of soil types. As in the case of the cone index-CBR relationships, these relationships are very similar for plastic soils but vary somewhat for non-plastic soils. However, for non-plastic soils, the variation of cone index at a constant North Dakota Cone bearing value is apparently not so great as the variation at constant CBR. This is shown by the following results:

Table VC-2

Variation of Cone Index of Granular Soils at Constant Cone Bearing Value

| Soil | Cone Index | North Dakota Cone Bearing Value |
|---|---|---|
| Shreveport Gravel | 45 | 100 |
| Shreveport Gravelly Sand | 40 | 100 |
| Fort Pierce Sand | 25 | 100 |
| Fort Pierce Silty Sand | 70 | 100 |

Furthermore, the agreement between the cone index - North Dakota Cone bearing value relationships for plastic soils is better than for the cone index-CBR relationships. In general, for the plastic soils, the cone index is about 60 to 75 percent of the North Dakota Cone bearing value.

g. Correlation of Cone Index and Unconfined Compression Strength

Unconfined compression tests cannot be performed on sands and gravels and are not truly indicative of the strength of silts. For plastic soils there has been found to be some general agreement between the cone index-unconfined compression strength relationships indicating that for these soils the cone index is numerically roughly four times the unconfined compressive strength measured in lb./sq.in.

h. Correlation of Cone Index with Liquid Limit and Soil Consistency

At the liquid limit all plastic soils have approximately the same shear strength. Hence, it would be expected that they would also have approximately the same cone index. Tests on nine soils for which the plasticity index was greater than 8 showed remarkable consistency in the cone index at the liquid limit, the value varying from 2 to 5. These data suggest that as a field expedient the liquid limit may be determined, with sufficient accuracy for trafficability purposes, by obtaining the water content at a cone index of about 3.

The range of water contents of fine grained plastic soils for which trafficability problems are encountered (muds) covers the range of cone index values from about 3, at or just above the liquid limit, to about 80 or more in the neighborhood of the plastic limit. The following descriptive adjectives have been proposed by the Corps of Engineers to designate the consistency of muds:

Table VC-3

| Cone Bearing: | Less than 15 | 15-35 | 35-50 | 50-80 | Above 80 |
|---|---|---|---|---|---|
| Adjective: | Fluid | Very Soft | Soft | Stiff | Firm |

i. Correlation of Cone Index with Trafficability of Fine Grained Soils

Investigations of the trafficability of fine grained soils (SF, ML, CL, OL, MH, CH, OH types) have indicated that these soils can be roughly divided into three main consistency ranges with respect to trafficability:

(i) Water content below the plastic limit; in this condition the soils exhibit semi-solid characteristics and do not present major trafficability problems.

(ii) Water content above the liquid limit; in this condition the soils exhibit semi-liquid characteristics and are so soft that bridging, roadways or other means of stabilization are necessary to permit the passage of vehicular traffic.

(iii) Water content between about the liquid and plastic limits; these soils are in the plastic range and are termed "mud". It is in this condition that the soils present serious problems in trafficability prediction.

Since the liquid limit of soil corresponds to a cone index of about 3 or 5 and the cone index at the plastic limit is usually 80 or more, it is in the range of cone index values between about 5 and 80 that trafficability problems are encountered in muds. Since a CBR of 3 is equivalent to a cone index of about 90 it will be seen that the stabilities of muds have as their higher values approximately the lowest CBR values which are usually measured.

Test data on the correlation of cone index with trafficability have been reported for five sites (VB-12 and 13):

(i) natural mud area at Camp Hulen; soil type ML

(ii) natural mud area at Yuma; soil type ML

(iii) artificially prepared mud area; soil type CL

(iv) artifically prepared mud area; soil type ML

(v) artificially prepared mud area; soil type CH

The following types of vehicles were used in the tests:

(i) $2\frac{1}{2}$, 4 and 6 ton trucks fitted with high and low pressure tires and both with and without traction devices.

(ii) D-7 and D-8 tractors

(iii) 6 ton Athey crawler wagons

(iv) Weasel M29C

(v) LVT(A)(4)

Photographs of these vehicles are shown in Figure VC-12.

In the investigations, the drawbar pull of the vehicle, the general performance of the vehicle, the depth of rut and the cone index and moisture content of the soil both before and during testing were recorded. In the earlier tests, traffic was often continued until 1000 passes had been completed but it became apparent that this number of passes was not necessary to determine if the vehicle was going to be immobilized. Usually immobilization occurred in a very few passes, if at all, and by about 40 passes it was evident whether traffic was improving soil conditions or making them worse.

A summary of the correlation between cone index and trafficability for the different vehicles and soil types is shown in Table VC-4. The cone index values in the table are those determined before traffic since it is from these values that a prediction of trafficability would be made. The data in the table leads to the following conclusions with regard to the trafficability of fine grained soils for the various vehicles:

TRACTORS

For tractors fitted with 60" extensions, a minimum cone index of about 15 is required for limited operations.

TABLE VC-4 - CORRELATION OF CONE INDEX AND SOIL TRAFFICABILITY

| Site | Soil Type | Tractor with 60" Extension | Tractor Without Extensions | Alley Wagons | Mud Sleds | Trucks (with and without traction devices | LVT-4 Tank | Weasel |
|---|---|---|---|---|---|---|---|---|
| Yuma | ML (natural) | Limited operation on soil with a C.I. of 12. | Operated on minimum C.I. or 25-50 | Without extension, wagons sank to firm bottom where C.I. was in excess of 55. | Operated on C.I. of about 10. | Limited operation on material with C.I. of 25-50. Layers must also be within ground elevation. | | |
| Hulen | ML (natural) | Limited opsration on soil with a C.I. of 15. | No tests | Operated on material with C.I. of 15 though wagons occasionally stuck in 6" or 7" rut. | Operated on C.I. of about 10. | Limited operation on material with C.I. of about 60. | | |
| Vicksburg | CL(artificial) | Limitsd operation on soil with a C.I. in the rangs of 12 to 20. | Limited operation on layer with C.I. of 30 to 35. | Limited operation, without extensions on material with C.I. of 35. | Operated on C.I. of about 10. | Operated on material with C.I. of 50. However, if layer is not within 10" of surface, trucks are liable to stick on first pass. | | |
| Vicksburg | ML (artificial) | | Soil with C.I. of 20 and 36 at depths from 6" and 12" is traffioable. | | | Soil with C.I. of about 40 is not trafficable. Soil for vehicle C.I. between depths of 6" and 12" varies from 35 to 65 or from 60-74 are on approximate boundary between limited and unlimited traffioability | Soil with C.I. of about 30 between 6" and 12" depth has very limited trafficability. Soil with C.I. of about 40 between 6" and 12" depth is traffioable. Soil with C.I. of 25 at 6" depth and 110 at 12" depth is traffioable. | Unlimited operation on soil with C.I. of about 10. Limited operation on soil with C.I. of about 7. Vehicle immobilized due to mud-packing. |
| Vicksburg | CH (artificial) | | Soil with C.I. of 33 and 44 at depths of 6" and 12" is traffioable. Limited operation on soil with C.I. of 25 and 30 at 6" and 12" depth. | | | Soil with C.I. over top 6" and reaching 100 at 12" depth is non-trafficable. Soil with C.I. of 50 and 90 at depths of 6" and 12" is traffioable. | Soil with C.I. of 20 and 30 at depths of 6" and 12" has limited trafficability. Soil with C.I. of 50 and 90 at depthz of 6" and 12" is trafficable. | Unlimited operation on soil with C.I. of about 20. Limited operation on soil with C.I. of about 8. Immobilization largely due to stickiness and mud-packing. |



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For tractors without extensions, a minimum cone index of about 30 in the upper 12" of soil is required for limited operation. In some cases unlimited operation may be possible on soils having a cone index of about 40.

ATHEY WAGONS

For Athey Wagons without extensions a minimum coneindex in the range 35 to 50 within about 4" of the surface is required for limited operation.

TRUCKS

There is no clear indication of the cone index required for operation of a 2½ ton truck. There is some evidence that a minimum cone index of about 50 to 60 is required for limited operation but the depth at which the required cone index occurs is also of importance. The effect of depth to the required cone index is an aspect of the problem which requires further investigation. In general, soils having a minimum cone index of about 50 at 6" depth and increasing to about 100 at 12" depth appear to be suitable for more than limited operation.

L.V.T. (A)(4)

The limited data available indicates that areas for which the upper 12" of soil has a cone index of about 30 are suitable for limited operation of L.V.T. (A)(4)s and that a minimum cone index of about 40 is required for unlimited operation.

WEASEL M29C

The Weasel can operate on soils of very low bearing capacity and in many cases has operated on soil that was too soft to support a man. Limited operation is possible on soils having a cone index of 7 or 8 and unlimited operation on soils having a cone index of about 10. However, the Weasel is very susceptible to immobilization due to mud-packing in the track mechanism.

The depth of the layer which has sufficient strength to support a given vehicle and the strength of the soil above the layer are important factors in determining trafficability. If the material above the supporting layer is soft it can be pushed out of the track of the vehicle relatively easily and the vehicle may sink considerably yet still be able to advance. For example, when the soft material above the supporting layer has a cone index less than 15, vehicles have been able to operate at depths equal to or sometimes greater than the ground clearance. This condition is particularly apt to occur when a very soft to fluid mud is underlain by a definite firm layer. However, if the material above the supporting layer is relatively stiff, the force required to displace it may be greater than that required to displace a soft material, even though the depth of sinkage is less.

In practically all cases investigated the cone index has increased with depth. Areas may also be encountered where there is a crust near the ground surface which has the required bearing value but the underlying soil is too soft to support traffic. Data are not available for this soil condition but it is considered necessary that there should be no decrease in bearing value below the firm layer for at least 18" in order for the requirements in the summaries to be applicable.

The drawbar pull which the vehicle can develop varies with the speed of the vehicle. It is well known that a vehicle can tow a greater load if it gets a good start on good ground and keeps moving than if it has to stop and start again. However, in the movement of military vehicles it cannot be assumed that they can traverse an area without stopping. Hence, the drawbar pull measured in the tests was that which a vehicle could develop when starting up from a stationary position.

An attempt has also been made by the Corps of Engineers to correlate the cone index with the drawbar pull which may be developed by various types of vehicles. This correlation would be useful in predicting the effects of gradients and tow-loads. Some difficulty was experienced in obtaining a satisfactory correlation and therefore, as an expedient in an early report (VB-12), conservative estimates were made of the minimum drawbar pull that could be expected for limiting cone index conditions. These estimates, expressed as ratios of drawbar pulls to weights of vehicles, for three different test sites are shown in Table VC-5.

The values in the table are stated to be conservative estimates and this is also indicated by subsequent data obtained at the Waterways Experiment Station (VB-13) for a tractor operating on a prepared clayey silt. The correlation of cone index with the ratio of drawbar pull/weight of vehicle in these tests is shown in Figure VC-13. It will be seen that the ratios are somewhat higher than the values indicated in Table VC-5.

It also seems likely that the drawbar pull will vary to some extent with soil type as well as with cone index. In silty soils the tractive force developed by friction will probably be greater than in plastic clays. This is also indicated by the data in Table VC-6, which although very limited in extent, shows the drawbar pull on a CH type soil to be consistently lower than for an ML soil with a similar cone index. Further data on this subject is required before any definite conclusions can be drawn.

Investigation of the effect of remoulding of the soil under traffic was made by comparing the cone index in the rut caused by the vehicle with the cone index of the soil at the same depth before traffic was applied. Analysis of the data for Camp Hulen, which was a natural area, showed that the cone index of the soil at the surfaceof the rut averaged about 40% less than the cone index before traffic. The data for the artificially prepared Vicksburg test areas showed no consistent trend. This indicates, as would be expected, that the remoulding effect of traffic causes a small loss of strength in natural soil deposits but has very little effect on artificially prepared soils which nave no structural or thixotropic strength.

### j. Correlation of Cone Index With Trafficability of Coarse-Grained Soils

Investigations of the trafficability of sandy beaches along the Pacific Coast of the United States and on Oahu, T. H. have been made by the University of California Institute of Engineering Research. In these studies the following characteristics of the beach were obtained:

(i) Ground photographs of the beach and surf together with vertical, into shore and alongshore aerial photographs of the beach.

(ii) A ground profile of the beach.

Table VC-5

Minimum Ratio of Drawbar Pull to Weight of Vehicle

| Vehicle | Cone Index | Yuma Test Site Soil Type ML (Natural) | Camp Hulen Test Site Soil Type ML (Natural) | Vicksburg Test Site Soil Type CL (Artificial) |
|---|---|---|---|---|
| 2½ ton truck with traction devices | 50 - 80 | 0.12 | 0.10 | 0.24 |
| 4 ton truck with traction devices | 50 - 80 | 0.11 | 0.10 | 0.23 |
| 6 ton truck with traction devices | 50 - 80 | 0.10 | 0.10 | 0.20 |
| Tractors with extensions | 15 - 35 | 0.2 - 0.4 | 0.36 | 0.2 - 0.4 |
| | 35 - 50 | 0.4 - 0.6 | | 0.4 - 0.6 |
| | 50 - 80 | 0.6 - 0.7 | | 0.6 - 0.7 |
| Tractors without extensions | 35 - 50 | 0.2 - 0.3 | | 0.2 - 0.3 |
| | 50 - 80 | 0.3 - 0.4 | | 0.3 - 0.4 |

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Table VC-6

Drawbar Pulls Developed on Different Soil Types

| Vehicle | Soil Type ML | | Soil Type CH | |
|---|---|---|---|---|
| | Cone Index | Drawbar Pull | Cone Index | Drawbar Pull |
| Tractor | 41 | 26,000 lb. | 25 | 13,500 lb. |
| | 28 | 16,000 lb. | 28 | 1,000 lb. |
| | 37 | 16,000 lb. | | |
| | 28 | 13,000 lb. | | |
| 2½ Ton Truck | 66 | 13,000 lb. | 65 | 6,500 lb. |
| | 70 | 11,500 lb. | | |
| | 38 | 12,200 lb. | | |
| | 41 | 8,900 lb. | | |
| | 33 | 7,100 lb. | | |
| Weasel | 16 | 3,800 lb. | 25 | 2,300 lb. |
| L.V.T. - 4 tank | 66 | 21,300 lb. | 62 | 15,000 lb. |
| | 37 | 26,000 lb. | 26 | 13,400 lb. |
| | 32 | 14,000 lb. | | |

(iii) The variation of cone index across the profile of the beach together with the times at which readings were made and notes on the tide stages, etc.

(iv) Grain size and shape of representative sand samples.

The performance of various vehicles on the beaches was also observed. Notes were made of vehicle performance on different sections of the beach and also when moving parallel to the surf or up the beach face. Particular attention was given to whether or not a vehicle could start from a dead stop at different points on the beach and the maximum velocities attained by vehicles at various points were recorded.

It was found from a few extensive tests that there was a characteristic variation of cone index across the beach profile. This variation has already been discussed and typical curves are shown in Figure VB-24, 25 and 26.

An attempt was also made to correlate the cone index measurements with the trafficability of the beaches. Test data showing the cone index measurements at certain critical sections of the beaches, the beach slope and a summary of the trafficability characteristics of the beaches is presented in Table VC-7. There is no apparent correlation between cone index and trafficability. This would seem to indicate that the cone penetrometer is not a satisfactory instrument for evaluating the trafficability of coarse grained soils.

The only definite trend shown by the data in the table is that trafficability increases with decreasing beach slope. This may be attributed to several factors:

(i) The flatter the slope, the more easily a vehicle can move up the beach face.

(ii) The flatter the slope, the greater the width of beach affected by tidal variations and the more extensive is the moist section which has the better trafficability.

(iii) The flatter the slope, in general, the smaller and more angular are the grains. The more angular the grains the higher is the angle of friction of the sand; and the smaller the grains, the greater is the apparent cohesion developed by water menisci in the voids. Furthermore, the smaller the grains the less pervious is the material and the greater is likely to be the increase in strength due to transient loading of the saturated sand.

The data obtained in these investigations also confirms many of the ideas presented in the previous discussion of factors affecting trafficability. The section dealing with the effect of vehicle speed on the trafficability of sands may be summarized as follows:

(i) The bearing capacity of a soil under transient loads is greater than for static loads; the more transient the load, the higher the bearing capacity.

(ii) The increase in strength of a compact saturated sand due to transient loading may be greater than the increase in strength

TABLE VC-7 - CORRELATION OF CONE INDEX WITH TRAFFICABILITY OF BEACH SANDS

| Beach | Cone Index at limit of beckrush | | | Cone Index at limit of uprush | | | Cone Index on Berm | | | Average Slope of Beach Face | Trafficability |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | 3"depth | 6"depth | 9"depth | 3"depth | 6"depth | 9"depth | 3"depth | 6"depth | 9"depth | | |
| Green Beach, Oahu, T.H. | 21 | 44 | 59 | 39 | 55 | 107 | 35 | 72 | 92 | | Too poor for wheeled vehicles other than DUKWs. DUKWs operated with great difficulty often becoming stuck in the sand. Tracked vehicles could operate by themselves but occasionally had trouble when under heavy load, as when trying to maneuver a loaded jahseay. Time after time anLCM would beach, lower its ramp and discharge a wheeled vehicle which would immediately become stuck in the sand and have to be pulled free by a tractor. |
| | 33 | 42 | 72 | 39 | 55 | 102 | 45 | 60 | 99 | $\frac{1}{6}$ or 9½° | |
| Blue Beach, Oahu, T.H. | 10 | 25 | — | 10 | 24 | 29 | | | | | |
| | | | | 10 | 20 | 25 | | | | | |
| Yellow Beach, Oahu, T.H. | 10 | 31 | 68 | 16 | 21 | 37 | | | | | |
| | | | | 12 | 29 | 72 | | | | | |
| Soldiers Club Beach Fort Ord, Monterey, California | 4 | 18 | 50 | 27 | 68 | 160 | 11 | 46 | 55 | | Too poor for wheeled vehicles other than DUKWs. DUKWs operated on the beach itself but had difficulties getting out of the surf. On a number of occasions the DUKWs had to be pulled up the beach face by a tractor at the end of a run through the surf. Tracked vehicles had no difficulty on the beach. The Jeep would not negotiate the beach; it could run down grade but at any rise at all became badly bogged down. Cone index values of the sand at locations where the Jeep stopped were: Moving across dry berm: 10 at 3" depth, 41 at 5" depth, 45 at 9" depth. Moving up beach face: 15 at 3" depth, 57 at 5" depth, 125 at 9" depth. |
| | 5 | 18 | 41 | 27 | 62 | 159 | 7 | 34 | 55 | | |
| | 5 | 50 | 48 | 25 | 64 | 164 | 7 | 30 | 59 | $\frac{1}{7.5}$ or 7½° | |
| | 2 | 12 | 41 | 20 | 43 | 86 | 11 | 37 | 55 | | |
| | 5 | 34 | 45 | 14 | 41 | 73 | 9 | 41 | 64 | | |
| | 4 | 28 | 37 | 11 | 56 | 80 | 9 | 37 | 68 | | |
| Point Reyes Beach, California | | | | 7 | 18 | 25 | 5 | 20 | 29 | | Wheeled vehicles could move around to some extent but had great difficulty getting up the beach face. A Jeep could move parallel to the beach in 1st gear at about 4 mph. but could not move up the beach face. A 1½ ton 6x6 Dodge truck moved up the beach face with 6 wheel drive, low range, 1st gear at 2 mph. but could not move if it had been loaded. The DUKW ran parallel to the beach at 10 mph. on the beach face and at 4 mph. on the dry berm. It could not move up the beach face unloaded, when the tire pressure was 15 psi, but moved up at 3 mph. when the tire pressure was reduced to 10 psi. It was considered that DUKWs would have difficulty moving out of the surf as at Soldiers Club Beach. |
| | 9 | 30 | 44 | 4 | 20 | 27 | | | | | |
| | 7 | 28 | 41 | 7 | 24 | 27 | | | | $\frac{1}{9.6}$ or 6° | |
| Tank Farm Beach, Monterey, California | 9 | 25 | 32 | 21 | 46 | 66 | 9 | 45 | 109 | | Wheeled vehicles operated with difficulty on the beach face when moving parallel to the beach but became stalled when headed up the beach face. DUKWs had little difficulty and could move out of the surf and up the beach face without assistance as long as the tire pressure was low (15 psi). A DUKW became stuck after standing at the limit of beckrush for a few minutes. Tractors could operate with ease. |
| | 7 | 20 | 45 | 23 | 57 | 75 | 11 | 48 | 55 | | |
| | 7 | 23 | 41 | 23 | 45 | 66 | 12 | 46 | 55 | $\frac{1}{11.2}$ or 5° | |
| South Beach, Moss Landing, California | 11 | 43 | 52 | 26 | 75 | 109 | 9 | 39 | 98 | | Wheeled vehicles could operate on beach face but had difficulty on the dry berm. The Jeep ran parallel to the beach at the limit of uprush at 30 mph. and could start from a stop with ease. The Jeep had trouble getting up the berm and required a running start to accomplish this successfully. It could start from a stop on the berm while parallel to the beach but would not start when heading up the beach. The DUKW had no trouble negotiating the beach and had no difficulty in starting from a stop at any point on the beach, when heading either parallel to or up the beach face. Tractors could operate with ease. |
| | 9 | 50 | 59 | 28 | 59 | 130 | 4 | 36 | 59 | | |
| | 11 | 39 | 50 | 32 | 52 | 115 | 5 | 37 | 107 | $\frac{1}{17.6}$ or 3½° | |
| Santa Margarita River Beach, Camp Pendleton, Oceanside, California | 5 | 23 | 68 | 5 | 48 | 94 | 7 | 21 | 53 | | A Jeep could operate with ease on the moist sand on the beach face. However, a Jeep had some difficulty in operating on the dry sand on the berm and became stuck on several occasions. After a heavy rain the Jeep had no difficulty operating on the berm. A 2½ ton 6x6 truck (unloaded) became stuck while trying to start up the moist beach face but could operate on the level on the dry berm. DUKWs and tractors had no trouble operating on either the moist face or the dry berm. |
| | 9 | 39 | 75 | 5 | 45 | 93 | 7 | 34 | 64 | | |
| | 5 | 50 | 75 | | | | 7 | 39 | 69 | $\frac{1}{23.0}$ or 2½° | |
| Clatsop Spit Beach, Oregon | | | | 11 | 50 | 100 | 14 | 62 | 112 | | Jeeps and DUKWs could travel up or along the moist beach face as fast as they could travel on a highway in high gear, high range. The Jeep could travel parallel to the beach at about 10 mph. on the dry sand. Even after the dry sand had been disturbed by tractors the Jeep could operate, start up when facing along or up the beach and even tow a loaded trailer under these conditions. DUKWs could also operate without difficulty on the dry sand. |
| | | | | 9 | 46 | 50 | 21 | 73 | 115 | | |
| | | | | 12 | 52 | 80 | 18 | 73 | 119 | $\frac{1}{58}$ or 1° | |

of a dry sand. If this is accepted as a tentative hypothesis it leads to the conclusions expressed in (3), (4) and (7) below.

(iii) For transient loads, the bearing capacities of the saturated and moist sections will be about the same, as will the bearing capacities of the dry and submerged sections; however, the maximum bearing capacity in the saturated and moist sections will be about twice the bearing capacity in the dry and submerged sections.

(iv) On a beach, the zone of maximum bearing capacity for transient loads will be closer to the saturated section than the zone of maximum bearing capacity for static loads.

(v) In the submerged section of a beach, and to some extent in the saturated section, the track or tire impressions of vehicles will tend to be eliminated by movement of sand into the impression.

(vi) When a vehicle drives up the beach face, the depth of track or tire impression will tend to increase progressively from the saturated to the dry section of the beach; the rolling resistance will increase with increasing depth of tire impression.

(vii) The bearing capacity of the dry section of a beach will be lower than that of any other section with the possible exception of the submerged section and in addition the rolling resistance of the dry section will be greater than for any other section. Hence, in general, the dry berm will be the least trafficable section of a beach.

Many of these effects were observed in the trafficability studies (VB-15). For example, the increased bearing capacity and smaller deformations of a soil under transient loads as compared with static loads are illustrated by the following incidents. At Tank Farm Beach, Monterey, California, a DUKW with low tire pressures could run out of the surf and up the beach face without assistance. However, when the DUKW was allowed to stand at the limit of backrush for a few minutes it settled appreciably and was unable to move forward again. The bogged-down DUKW is shown in Figure VC-14a. A similar incident occurred on the sandy section of Cobble Beach, Spanish Bay, Monterey Peninsula, California where a DUKW, driven in the uprush, sank four inches when it was stopped. However, in this case the DUKW was able to start up and move forward again without assistance.

The variation of bearing capacity for transient loads across the profile of a beach could not easily be measured directly. However, for vehicles moving parallel to the beach, greater bearing capacity is reflected by improved trafficability and an increase in the speed at which the vehicle can be driven. Hence, the variation of bearing capacity for transient loads in the different sections of a beach is indicated by the following results of trafficability studies at South Beach, Moss Landing, Monterey Bay, California:

| Vehicle | Section of Beach | Trafficability Notes |
|---|---|---|
| Jeep | Limit of uprush | Ran parallel to beach at 30 mph in third gear, high range and in four wheeled drive |

| Vehicle | Section of Beach | Trafficability Notes |
|---|---|---|
| Jeep | Upper limit of moist sand | Ran parallel to beach at 22 mph in third gear, low range and in four wheeled drive. |
| Jeep | Old high water line | Ran parallel to beach at 18 mph in second gear, low range and in four wheeled drive. |
| Jeep | Dry berm | Ran slowly and with some difficulty parallel to the beach. |

This data is summarized and a profile of the beach is shown in Figure VC-15. It is unfortunate that no data was obtained for the moist section of the beach face. However, the investigator also noted that "the velocity with which a wheeled vehicle could travel was found to be low in the dry sand berm, increasing until wet sand was reached and then remaining about constant right down to saturated sand in the uprush during a low tide". Hence, these results are a good indication that for transient loads, the bearing capacity of the saturated section of a beach is approximately equal to the maximum bearing capacity in the moist section and also, that the bearing capacity decreases progressively from the saturated to the dry sections of a beach.

The relatively poor bearing capacity of the dry and submerged sections of a beach as compared with that of the moist section was also observed at Soldiers Club Beach, Fort Ord, Monterey, California. Here a DUKW would run on the moist section in fourth gear with a tire impression about 3/4" in depth; on the dry section it had to be run in second gear and the depth of tire impression was 3-3/4". However, although the DUKWs would run satisfactorily once they were on the beach, some difficulties were encountered in getting them out of the surf and on several occasions a DUKW had to be pulled up the beach face out of the surf by a tractor. This operation is shown in Figure VC-14b, while the relatively shallow tire impression on the moist sand is shown in Figure VC-14c.

The variation in transient bearing capacity across a beach profile will also be indicated by the depth of tire impression in the sand, although in the saturated section, the residual depth of impression will probably be somewhat less than the actual impression. Measurements of the depth of tire impression left by a jeep at various sections of the profile of Santa Margarita River Beach are shown in Figure VC-16. The depth of impression increases progressively from about $\frac{1}{2}$" in the saturated section to about 3 inches in the dry section. Cone penetrometer readings were also taken at depths of 3", 6" and 9" at points where the depth of tire impression was measured. The variation of cone index across the profile is shown in Figure VC-16. It will be seen that the cone index (or the static bearing capacity) is a maximum at some zone in the moist section of the beach and the cone index in the saturated section is considerably lower, in contrast to the distribution of bearing capacity indicated by the depth of tire impressions. Hence, these results indicate that a saturated soil having a relatively low bearing capacity under static loads may have a relatively high bearing capacity under transient loads.

This same result is shown by the test data obtained at Santa Margarita River Beach and presented in Figure VC-17, which shows the variation of cone index and maximum jeep velocity parallel to the beach at different points across

the beach profile. It will be seen that the static bearing capacity, as measured by the cone index, is fairly low in the dry section, increases to a maximum in the moist section and then decreases again in the saturated and submerged sections. However, the transient bearing capacity, as measured by the maximum jeep velocity parallel to the beach, increases much more than the static bearing capacity from the dry to the moist sections of the beach and increases still more in the saturated section. In this case the increase in maximum jeep velocity corresponding to a decrease in cone index of the soil is clearly shown.

The possible difference between the bearing capacity of a soil under static and transient loads illustrates one of the limitations of the cone penetrometer for trafficability measurements. The cone index is a measure of the bearing capacity under static loads whereas the stresses in a soil caused by moving vehicles are of transient duration. If the degree of saturation of a soil does not vary too greatly, the effect of transient loading would be about the same for all soil conditions and it would be possible to correlate trafficability with cone penetrometer readings. However, this correlation would not necessarily be true for the same soil with a radically different degree of saturation. This difficulty could, of course, be overcome by correlating cone index and trafficability for different moisture conditions of the soil. It is, perhaps, because the soil moisture has not been given full consideration that some difficulty has been experienced in the past in attempting to correlate the cone index values for sands with their trafficability characteristics.

An argument in favor of the cone penetrometer is that during any operation, no guarantee can be given that a vehicle will not stop for some reason or other. In the case of a stationary vehicle starting up, the static bearing capacity would be one of the controlling factors. This possibility must evidently be taken into account in selecting an instrument for bearing capacity measurements. Furthermore, trafficability estimates based on cone penetrometer measurements would be satisfactory for static loads and conservative for transient loads whereas an instrument designed to estimate trafficability by applying a transient load to a soil would be satisfactory for moving wheel loads but on the unsafe side for stationary vehicles.

On all beaches investigated, the great difference between the trafficability of moist or saturated sand and the trafficability of dry sand was observed. This difference is due to the lower bearing capacity of dry sand, resulting in greater deformation under wheel loads, a greater depth of tire impression, greater rolling resistance and, hence, poorer trafficability. For example, at Clatsop Spit Beach, Oregon, a DUKW could travel parallel to the beach or up the beach face on moist sand in high gear, high range and with tires inflated to 40 psi as fast and as easily as it could travel on a highway. However, with a tire pressure of 40 psi on the dry sand it could only travel at 3 mph in low gear, low range and with six-wheeled drive. When the tire pressure was reduced to 20 psi the DUKW could travel at 4 mph in second gear, high range. The considerable difference between the depth of tire impressions caused by a DUKW on the wet or moist sand of the beach face and on the berm sand, even after it had been moistened by a heavy rain, is shown in Figure VB-14. Under normal conditions the berm sand would be dry and the depth of tire impression would be greater than that shown.

k. Table of Trafficability of Major Soil Types

On the basis of the data obtained in the earlier tests conducted by the U. S. Corps of Engineers (VB-12) together with the observations and experi-

ence of the personnel conducting the tests, a table was prepared showing the relationship of water content, cone index, slipperiness and stickiness of different types of soils to the mobility of standard military vehicles. In this table soils were divided into two main groups - coarse grained and fine grained - and fine grained soils were further subdivided on the basis of plasticity index. Columns were provided showing what was considered to be the approximate correlation between water content (expressed in percent above plastic limit), cone index (called cone bearing value) at a depth of 10 inches, degree of slipperiness and stickiness, the class of vehicle that could operate on the soils, the drawbar pull that vehicles could be expected to develop and the maximum gradient they could traverse. This chart is shown in Table VC-8.

Vehicles were classified in five groups, designated by the letters A to E in order of decreasing mud mobility. This classification, as presented in the original report, is shown on Table VC-9.

It was emphasized in the report, that the trafficability table covers soils and vehicles on which tests had not been performed and its accuracy had still to be established. Nevertheless, it represented the most reliable information that could be obtained in the limited time available and was considered sufficiently accurate to predict the trafficability of soils in broad brackets.

Subsequent tests have indicated that the trafficability of coarse grained soils cannot be satisfactorily evaluated by the cone index, and the section of the table dealing with these types of soils cannot be considered reliable. However, more recent tests on the fine grained plastic soils have confirmed the possibility of correlating the cone index of these soils with their trafficability and this section of the table, modified by the correlations previously presented, may be considered to give as indication of the trafficability of the various soil types. Further test data is required before a more satisfactory correlation can be established.

1. Procedure for Prediction of Trafficability by Empirical Methods

In using correlations of the type presented in Table VC-8 for predicting the trafficability of soils, all pertinent information which may be conveniently obtained from a study of the area should be considered. The following is an extract from a report by the Waterways Experiment Station (VB-12), describing the data which should desirably be obtained and a proposed procedure for trafficability prediction.

"In the prediction of trafficability of areas, a survey of the route traversed is necessary. This survey should be conducted or at least supervised by a qualified soils engineer in conjunction with experienced equipment men. The data and samples required include:

(i) Representative cone index values to a depth of at least 3 ft. The values should be obtained for the undisturbed material and if possible for the remoulded material. Remoulding can be accomplished by tamping with bars or poles or by men tramping over a small area.

(ii) Representative samples for water content determination at the same locations and depths at which the cone index values were obtained.

# TABLE VC-8

## MOBILITY OF STANDARD MILITARY VEHICLES ON MAJOR SOIL TYPES

Based on Limited Operation (Approximately 25 Passes) Over Muddy Terrain

| Soil Classification | Typical Names of Soils in Group | Liquid Limit | Plastic Range (PI) | Percent Moisture above Plastic Limit producing Min. Cone Bearing Indicated | Approximate[1] Cone Bearing within 10 Inches of Ground Surface | Sticky | Slippery[2] for Equipment that can Operate | Class of Equipment[3] that can Operate: Good | Doubtful | No-Go | Estimated Drawbar Ratio to Total Weight of Vehicle: Type A | Type B | Type C | Maximum Slope Vehicle[5] can Traverse - in Percent: Vert. rise, ft / Horiz. dist., ft x 100: Type A | Type B | Type C | Remarks |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **COARSE-GRAINED MATERIALS** | | | | | | | | | | | | | | | | | |
| Gravel & gravelly soils GW, GC, GP | Well graded gravel & gravel-sand mixtures, little or no fines, well graded gravel-sand-clay mixtures excellent binder, poorly graded gravel & gravel-sand mixtures little or no fines | Not significant | Not significant | Not significant | 80+ | No | No | ABCDE | - | - | Estimated to be equal to or higher than values listed below for low PI materials. | | | | | | Not considered as sand. Highly stable under all conditions. |
| GM | Gravel with fines, very silty gravel, clayey gravel, poorly graded gravel-sand-clay mixtures | Plasticity must be determined for material passing 40-mesh screen. Detrimental only when PI exceeds 6. | | | 50-80 | May be | No | ABCD | | E | Estimated to be equal to or higher than values listed below for low PI materials. | | | | | | In general, vehicles without traction devices can operate on this material. Considered as very unlikely condition. |
| | | | | | 35-50 | slightly | No | AB | CD | E | | | | | | | |
| | | | | | 15-35 | " | No | A | B | CDE | | | | | | | |
| Sands & sandy soils SW, SC, SP, SM | Well graded sands & gravelly sands, little or no fines, well graded sand-clay mixtures excellent binder, poorly graded sands little or no fines, cohesionless sands with fines | Not significant | Not significant | Moist to saturated | Not applicable | May be slightly | No | AB DP | C | - | Estimated to be equal to or higher than values listed below for low PI materials. | | | | | | * For trucks with low-pressure tires only. |
| | | | | Dry | Not applicable, loose | | | A DP | B | CE | | | | | | | * For trucks with low-pressure tires only. |
| SM | Sands with fines, very silty sands | 20-30 | 6-15 | 2-4 | 50-80 | May be | No | ABC | D | E | Estimated to be equal to or higher than values listed below for low PI materials. | | | | | | Sands with fines having little or no plasticity should be included in SW, SC, SP group. |
| | | | | 4-8 | 35-50 | slightly | No | AB | C | DE | | | | | | | |
| | | | | 8-15 | 15-35 | " | Slightly | A | B | CDE | | | | | | | |
| **FINE-GRAINED MATERIALS** | | | | | | | | | | | | | | | | | |
| Feebly plastic ML, CL, ML, CL | Silts & very fine sands, sandy clays, silty clays, loam clays, peats | Not significant | 0-6 | 0-2 | 50-80 | No | No | ABC | D | E | 0.7 | 0.5 | 0.3 | 55 | 40 | 25 | |
| | | | | 2-4 | 35-50 | Slightly | Slightly | AB | C | DE | 0.6 | 0.4 | - | 30 | 30 | - | |
| | | | | 4-6 | 15-35 | Slightly | Slightly | A | B | CDE | 0.4 | - | - | 30 | - | - | |
| Moderately plastic soils ML, CL, OL, MH | Organic & inorganic silts, sandy clays, loam clays, silty clays | 25-40 | 6-12 | 2-4 | 50-80 | No | No | ABC | D | E | 0.6 | 0.4 | 0.2 | 30 | 30 | 15 | |
| | | | | 4-8 | 35-50 | Slightly | No | AB | C | DE | 0.5 | 0.3 | - | 40 | 25 | - | |
| | | | | 8-12 | 15-35 | Moderately | No | A | B | CDE | 0.3 | - | - | 25 | - | - | |
| Plastic soils CL, OL, CH | Organic and inorganic clays, silty clays, loam clays | 30-60 | 12-20 | 2-4 | 50-80 | No | No | ABC | D | E | 0.5 | 0.4 | 0.1 | 40 | 30 | 10 | |
| | | | | 4-8 | 35-50 | Slightly | No | AB | C | DE | 0.4 | 0.3 | - | 30 | 25 | - | |
| | | | | 8-12 | 15-35 | Moderately | No | A | B | CDE | 0.2 | - | - | 15 | - | - | |
| Highly plastic soils CH, OH | Organic & inorganic clays | 40-70 | 20-30 | 4-10 | 50-80 | No | No | ABC | D | E | 0.4 | 0.3 | 0.1 | 30 | 25 | 10 | |
| | | | | 8-12 | 35-50 | No | No | AB | C | DE | 0.3 | 0.2 | - | 25 | 15 | - | |
| | | | | 8-24 | 15-35 | Mod.-highly | No | A | B | CDE | 0.2 | - | - | 15 | - | - | Sticky condition both on bottom & overlying soil may immobilize equipment. |
| Very highly plastic soil CH | Clay | 50+ | 30+ | 8-20 | 50-80 | No | No | ABC | D | E | 0.3 | 0.2 | 0.05 | 25 | 15 | 5 | |
| | | | | 12-25 | 35-50 | Moderately | No | AB | C | DE | 0.2 | 0.1 | - | 15 | 10 | - | Same as above. |
| | | | | 20-40 | 15-35 | Highly | No | A | B | CDE | 0.1 | - | - | 10 | - | - | Probable that sticky condition will immobilize equipment. |

NOTES:
1. Where sand above fine layer has a cone bearing value of less than about 15, vehicles may operate at depths equal to and slightly greater than the ground clearance.
2. All conditions above the plastic limit are slippery for rubber tires. Statements pertaining to slipperiness apply only to track type vehicles with cleats or extensions and wheel vehicles with chains.
3. For cone bearing values less than about 15, bridging or roadways are required. See Appendix D for equipment classification.
4. Applies to operator tractors and trucks which were the basis of estimate.
5. Vehicles always without any towed equipment, assuming adequate power to cause slipping of tracks or wheels.
6. Cone bearing values are given adjective classifications as follows: 0-5 = fluid; 15-35 = very soft; 35-50 = soft; 50-80 = stiff; and 80+ = firm bottom.



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TABLE VC-9 - MOBILITY OF STANDARD MILITARY VEHICLES

1. General. The trafficability of military vehicles is dependent upon the physical characteristics of the vehicle. In the case of self-propelled vehicles, the basic factors that must be considered are unit ground pressure, ground clearance, traction and power for propelling the vehicle. Basic factors to be considered for towed vehicles are rolling resistence, axle loadings, ground clearence, gross weight with load, and unit ground pressure.

2. Mud Class. Mud class is that group of vehicles and vehicle types that have basic characteristics that give them approximately the same mobility in mud terrain. Only limited test data on a few specific items have been available for determining mud mobility of vehicles. However, in order to make a classification that incorporates all representative military vehicles, a paper study has been made of information in TM-9-2300, "Standard Military Motor Vehicles", TB 5-9720-1, "Technical Data for Grading, Excavating and Earth Moving Equipment", and other available literature on military vehicles and equipment. Since specifications alone do not reveal the true performance characteristics of vehicles, the classification listed below cannot be considered conclusive but rather as a guide until such time as detailed investigations and tests can be conducted to obtain actual performance data on all vehicles when operating in mud terrain.

| a. Class A. Negotiate a limited number of trips (not more than 25) over mud terrain having a minimum cone bearing value of 15 within a depth not to exceed 10 inches. | b. Class B. Negotiate a limited number of trips (not more than 25) over mud terrain having a minimum cone bearing value of 35 within a depth not to exceed 10 inches. | c. Class C. Negotiate a limited number of trips (not more than 25) over mud terrain having a minimum cone bearing value of 60 within a depth not to exceed 10 inches. | d. Class D. Capable of off-road travel over terrain having a cone bearing value in excess of 80 within a few inches of the ground surface. This type of terrain is not considered a mud condition. | e. Class E. Vehicles and other equipment which due to excessive weight, high unit ground pressure, lack of practice, lack of power, and other peculiarities of design, are not suitable for off-road use. |
|---|---|---|---|---|
| (1) Self-propelled vehicles | (1) Self-propelled vehicles | (1) Self-propelled vehicles | (1) Self-propelled vehicles | (1) Self-propelled vehicles |
| Carrier, light cargo, T-15<br>Carrier, personnel, 1/4 ton, M29C<br>Tractor, Crawler, Diesel, all types when equipped with 60-inch track extensions<br>Special low ground pressure D-7 tractor<br>Tractor, Crane, all types when equipped with 60-inch track extensions. | Landing vehicles, tracked, all types<br>Tank, light, T9E1<br>Tractor, Crane, all types, without track extensions<br>Tractor, Crawler, Diesel, all types, without track extensions | Carrier, cargo, T-14, with grousers<br>Carriage, motor, full-track all types, with grousers<br>Car, armored, 6x6, all types, with traction device<br>Tanks, all types, equipped with grousers<br>Tractors, high speed, all types<br>Trucks, 6x6, all types, with traction devices<br>Trucks, 4x4, all types, with traction devices<br>Vehicle, tank recovery, all types, with grousers | Carrier, cargo, T-14, without grousers<br>Carriage, motor, full-track, all types, without grousers<br>Car, armored, all types, without chains<br>Graders, road, motorized, all types<br>Shovels, crawler mounted, all types<br>Tanks, all types, without grousers<br>Tractors, wheeled, all types<br>Trucks, 4x4 and 6x6, all types, without traction devices<br>Vehicles, half-track, all types. | Bus, commercial type, all sizes<br>Cars, passenger, all types<br>Motorcycles, all types |
| (2) Towed vehicles | (2) Towed vehicles | (2) Towed vehicles | (2) Towed vehicles | (2) Towed vehicles |
| Sled, cargo, 1-ton, M-1<br>Sled, cargo, 4-ton, Engineer Board Spec. | No towed vehicles in this class | Crane, tractor operated, all types<br>Trailer, track laying, all types, with 60-inch track extensions<br>Trailer, wheeled, 1/4 to 1-ton capacity, all types<br>All trailer mounted equipment with gross weight of 2-1/2 tons or less | All trailer mounted equipment with gross weight less than 2 tons<br>Trailer, track laying, all types, without track extensions<br>Trailer, wheeled, all types, less than 8 ton capacity<br>Semi-trailers, all types, less then 8 ton capacity<br>Wagons, crawler, rock or dirt, all types | All trailer mounted equipment with gross weight of 8 tons or over<br>Trailers, wheeled, all types, with capacity of 8 tons and over<br>Semi-trailers, all types with capacity of 8 tons and over |

NOTE: This appendix was prepared by Captain W. R. Wallace of The Engineer Board

(iii) Representative samples of sufficient size for laboratory tests.

(iv) Estimate in the field the limiting area to which a group of representative cone index values, water contents and samples would apply.

(v) If areas are encountered where the cone index values are considerably below the general average, the extent of the area should be outlined and alternate routes selected and explored.

(vi) Depth to ground water table.

(vii) Information on terrain that will determine whether vehicles can be deployed over a wide area or whether they must be concentrated in a few sets of ruts.

(viii) If area is covered by water, determine if drainage is desirable (for limited operation) and practical (for unlimited operation).

(ix) Determine if area is subject to flooding from natural causes and if area can be flooded by enemy or friendly forces.

(x) Rough estimates of the maximum and average slopes.

(xi) Determine entrance and exit conditions for muddy areas.

(xii) Note and sample sand areas and investigate possibility of keeping such areas wetted.

(xiii) Vegetation along route, particularly grass or other vegetation that will form a mat.

(xiv) Laboratory tests on representative samples for liquid and plastic limits and water contents on all samples taken at the locations of cone penetrometer readings.

After the above data is assimilated it should be evaluated. Scattering of the data will be found which will be caused both by variations in soil conditions and limitations of the instrument used. In general, the data should be evaluated on the limiting case, that is, a low average value. Other conditions such as elevation of water table and covering of vegetable mat should be considered in evaluating the data.

"After the limiting conditions are selected, the soil classification, liquid limit, plastic limit, water content above the plastic limit, and cone index should be compared with the entries in the trafficability table and the type or types of vehicles selected that can be expected to negotiate the area under consideration. From the other columns in the table the degree of slipperiness, stickiness, estimated drawbar pull and estimated maximum gradient can be determined for the limiting conditions."

m. Summary of Trafficability Investigations

The main factors affecting the trafficability of soils are the soil

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type, bearing capacity, stickiness, slipperiness, the slope of the area and the density of traffic. Indirectly, the water content is an important factor since for fine grained soils, the water content determines the strength, slipperiness and stickiness.

Two methods have been used to predict the trafficability of soils: the rational approach to the problem and the empirical approach.

In the rational approach, the appropriate soil properties are measured and are then used to evaluate the behavior of the soil under traffic loads. An apparently satisfactory instrument, the soil truss, has been developed to measure the strength characteristics of soil in the field but an adequate theory for the mechanics of soil under wheel or track loads is required in order to determine the depth of sinkage and rolling resistance of the vehicle.

In the empirical approach a penetrometer test is used as an indication of the bearing capacity and traction capacity of the soil. The resistance to penetration of different soils by a standard instrument (the cone index) is correlated with the trafficability of the soils and the correlation is then used to predict the trafficability of other areas. Investigations have shown that this method is apparently satisfactory for fine grained soils, but it has not been found possible to determine a correlation for coarse-grained soils. This latter difficulty is probably due to the unsuitability of the cone penetrometer for determining the bearing capacity of sands and gravels. For these types of soils the bearing capacity is greatly affected by the confining pressures on the soil. Under vehicle loads, confinement is produced by the width of the tires or tracks but no confinement is used in the cone penetrometer test.

Further investigations of both the rational and the empirical methods of determining soil trafficability are required before a satisfactory method or methods for all types of soils can be developed.

From the investigations conducted to date, the following general conclusions can be drawn:

(1) Fine Grained Soils: These soils present trafficability problems when the water content is sufficiently high that the soils are termed "muds". Correlation tables have been developed for the predicting of trafficability of muds within broad limits. For each type of vehicle there is a certain cone index requirement for vehicle mobility. The requirement depends on the depth of the supporting layer, the strength of the soil above this layer, the slipperiness and the stickiness. It has been found that the Weasel can operate on extremely soft muds having a cone index as low as 10 but is very susceptable to immobilization by soil jamming in the track mechanism.

(2) Coarse Grained Soils: With regard to the trafficability of sandy beaches it has been found that the flatter the beach, the better is the trafficability. It has also been shown that these soils have sufficient bearing capacity and tractive properties for trafficability provided the soils are confined. This confinement can often be achieved by the use of wide tracks or large low pressure tires.

The trafficability of wet sand is also better than that of dry sand. From this point of view it is desirable that amphibious craft should discharge their vehicle cargo during low tides so that use may be made of the moist section of the beach face which is exposed during low tides. Further, landings



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made shortly after a heavy rain will encounter better trafficability on the normally dry berm-sand.

On many beaches tractors can operate while wheeled vehicles cannot. However, where both types of vehicles can operate, tractors tend to disturb the surface and make subsequent operations with wheeled vehicles very difficult, if not impossible, on all but very flat beaches. During landing operations on all steep and most moderate beaches it is desirable that tracked vehicles should be present to pull wheeled vehicles immediately away from landing craft as they unload.

On sandy beaches, as well as muds, the Weasel has been found to have the best trafficability characteristics of any vehicle investigated. It would be advantageous, therefore, to equip a Weasel for use by the Beach Master (or any other person who needs to travel a beach considerably during a landing operation) rather than a wheeled vehicle.

3. References

V C-1: Army Operational Research Group Report 238 (British Army)

V C-2: Boyd, Keith - "Suggested Method of Test for Bearing Power of Soil by Means of a Cone Penetrometer" - Procedures for Testing Soils, American Society for Testing Materials, September 1944, pp. 117-118.

V C-3: Joint Army - Navy Experimental Testing Board (JANET) - "Beach Trafficability Unit Final Report" - September 1945 (RESTRICTED: by letter CNO ser 002629 - p. 32 to CNR)

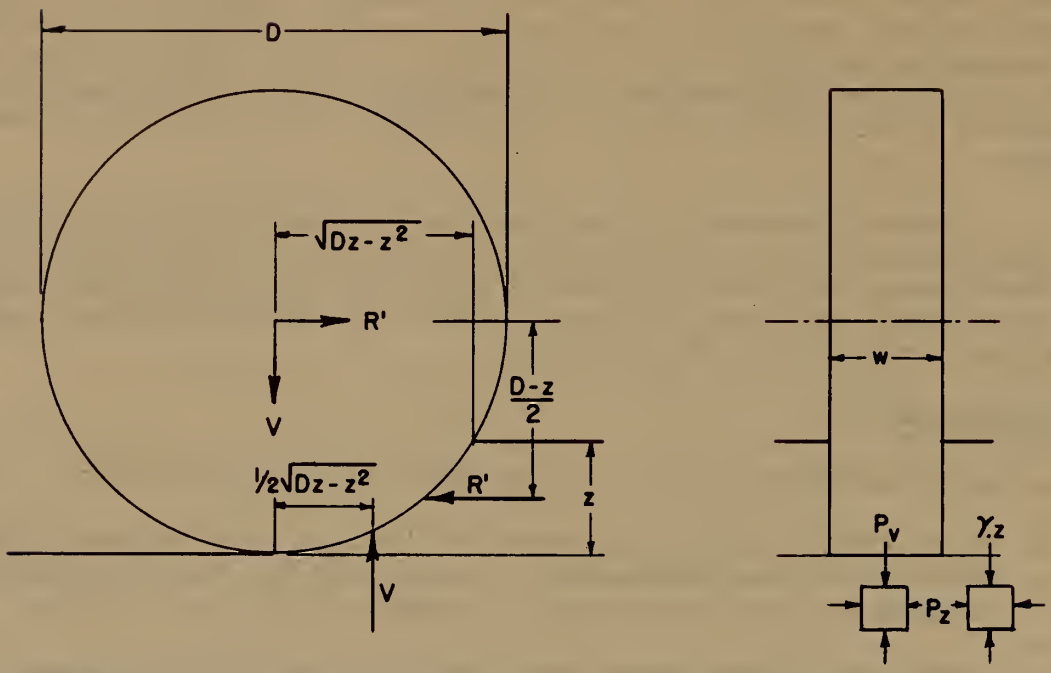



Figure V C-1.

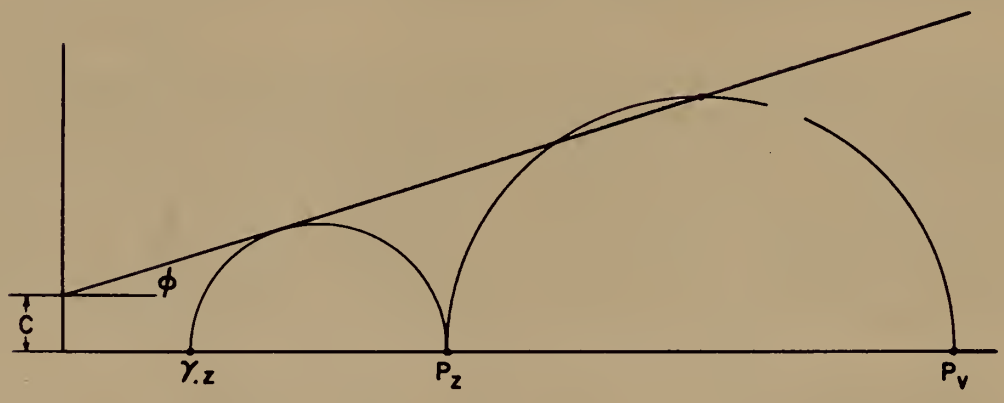



Figure V C-2.



FIGURE VC-3 - WEISS USING TRUSS

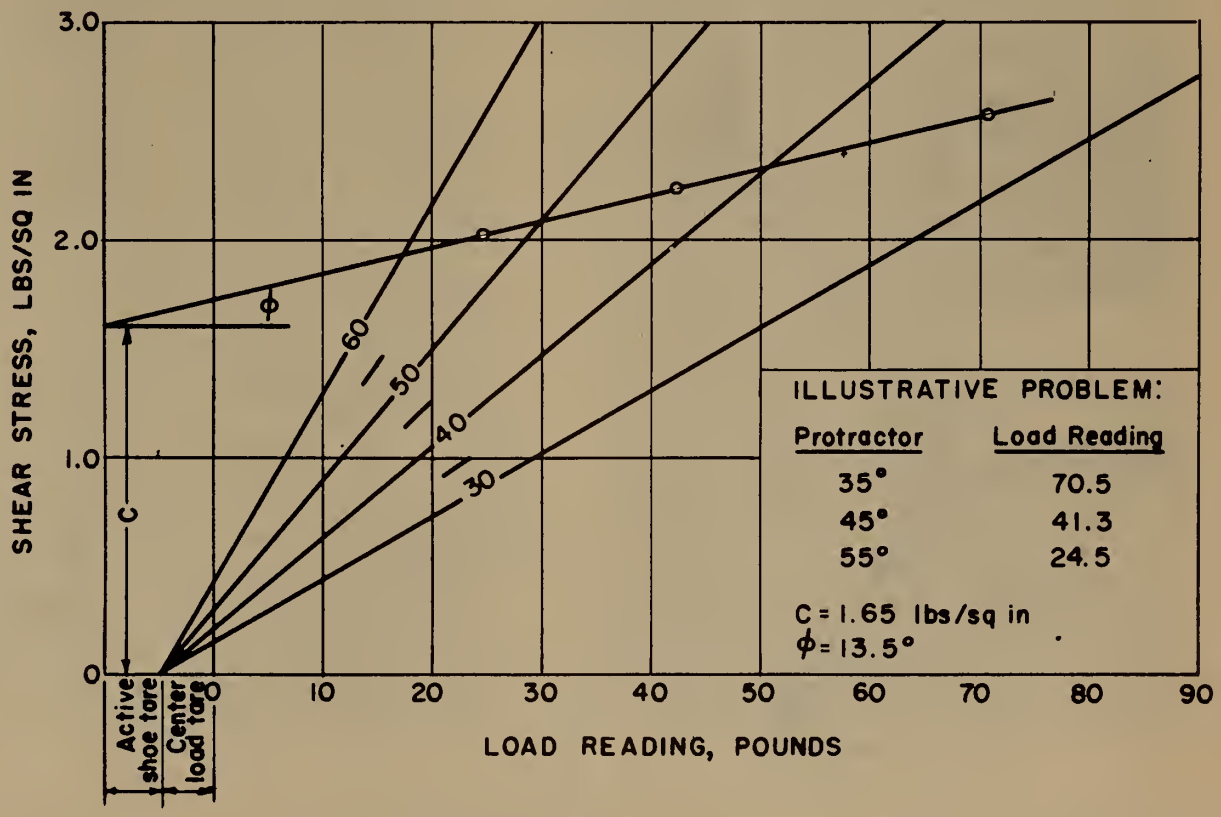
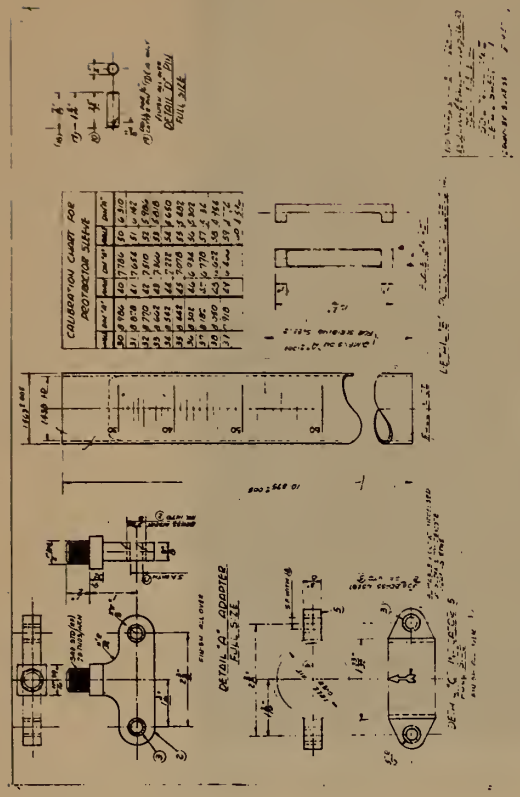
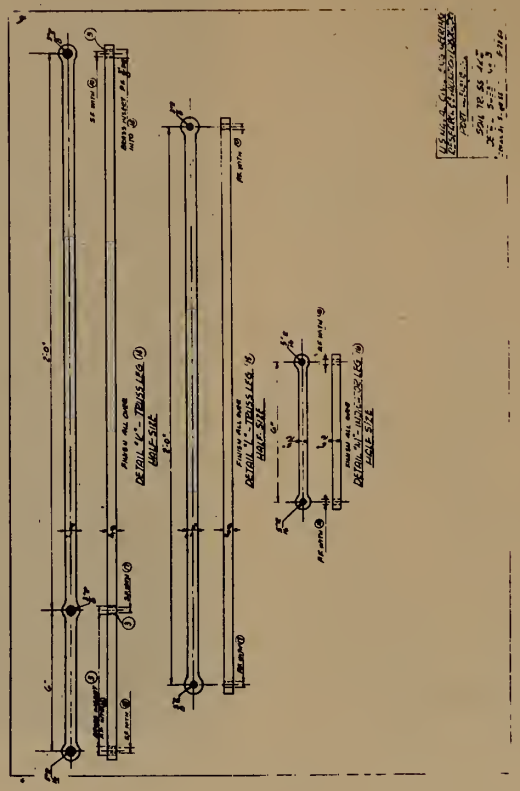
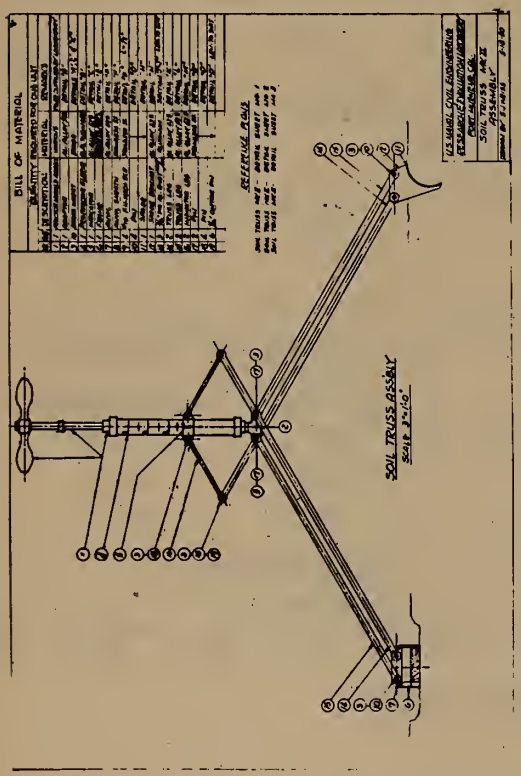



FIGURE VC-4 - SOIL TRUSS MARK II FIELD CHART







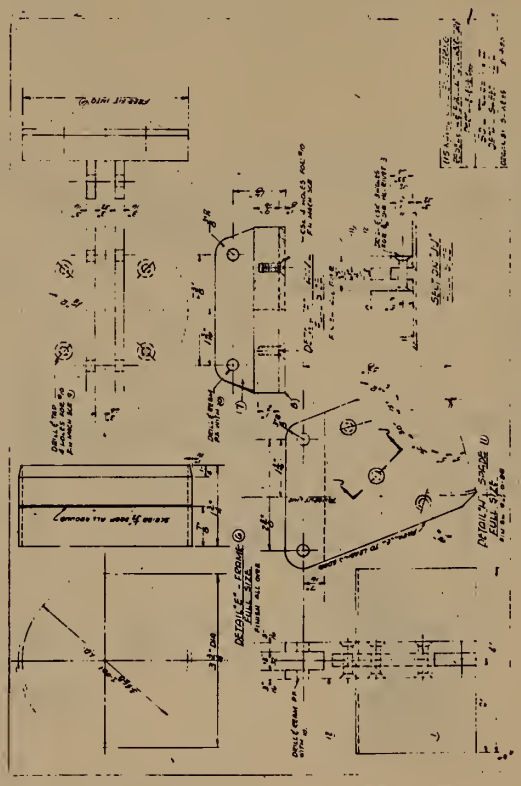


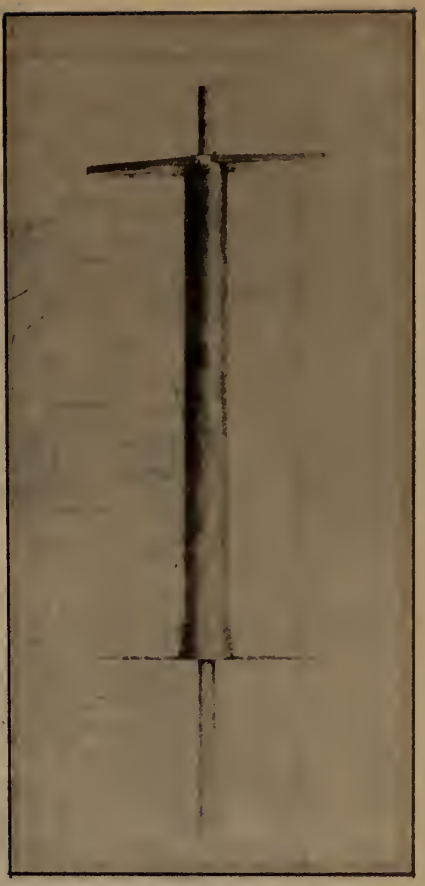
FIGURE VC-5 - DETAILS OF TRUSS



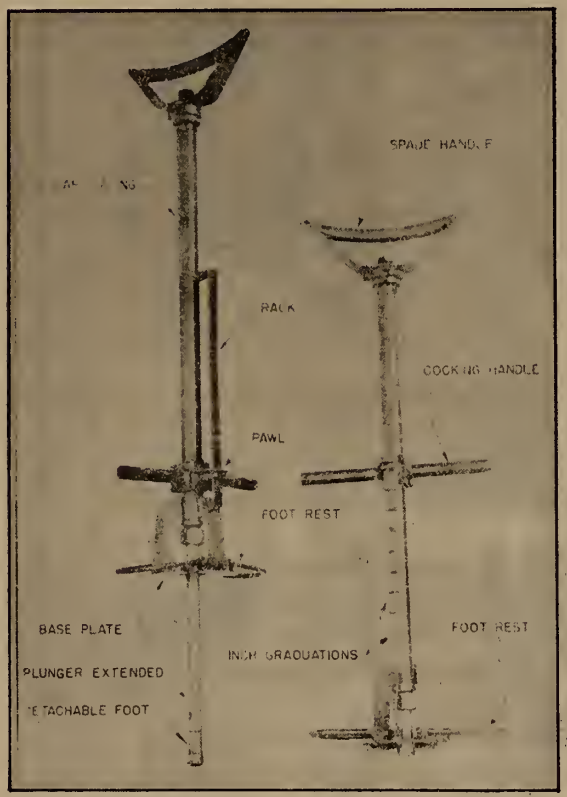
a. North Dakota Cone Apparatus



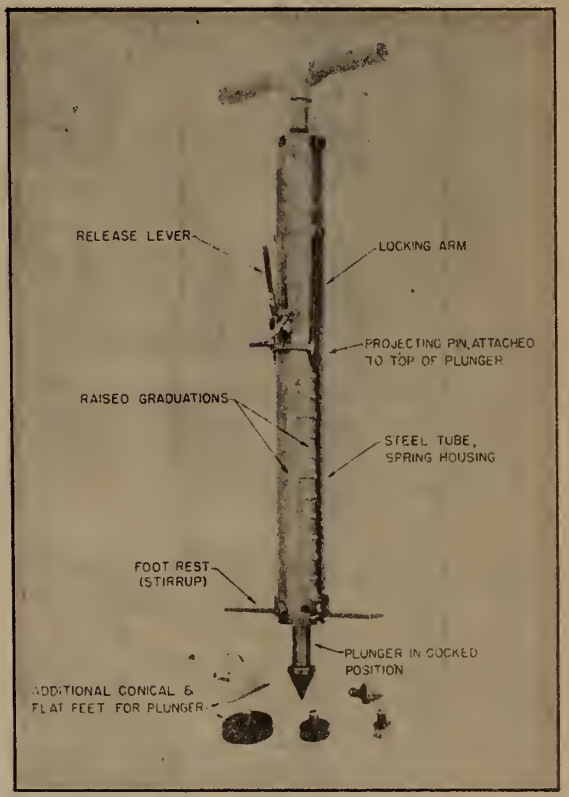
b. Cone Penetrometer - Corps of Engineers



c. BTU Beach Tester Model 2




d. Malcom Campbell Beach Tester




e. BTU Beach Tester Model I

Figure V C-6.

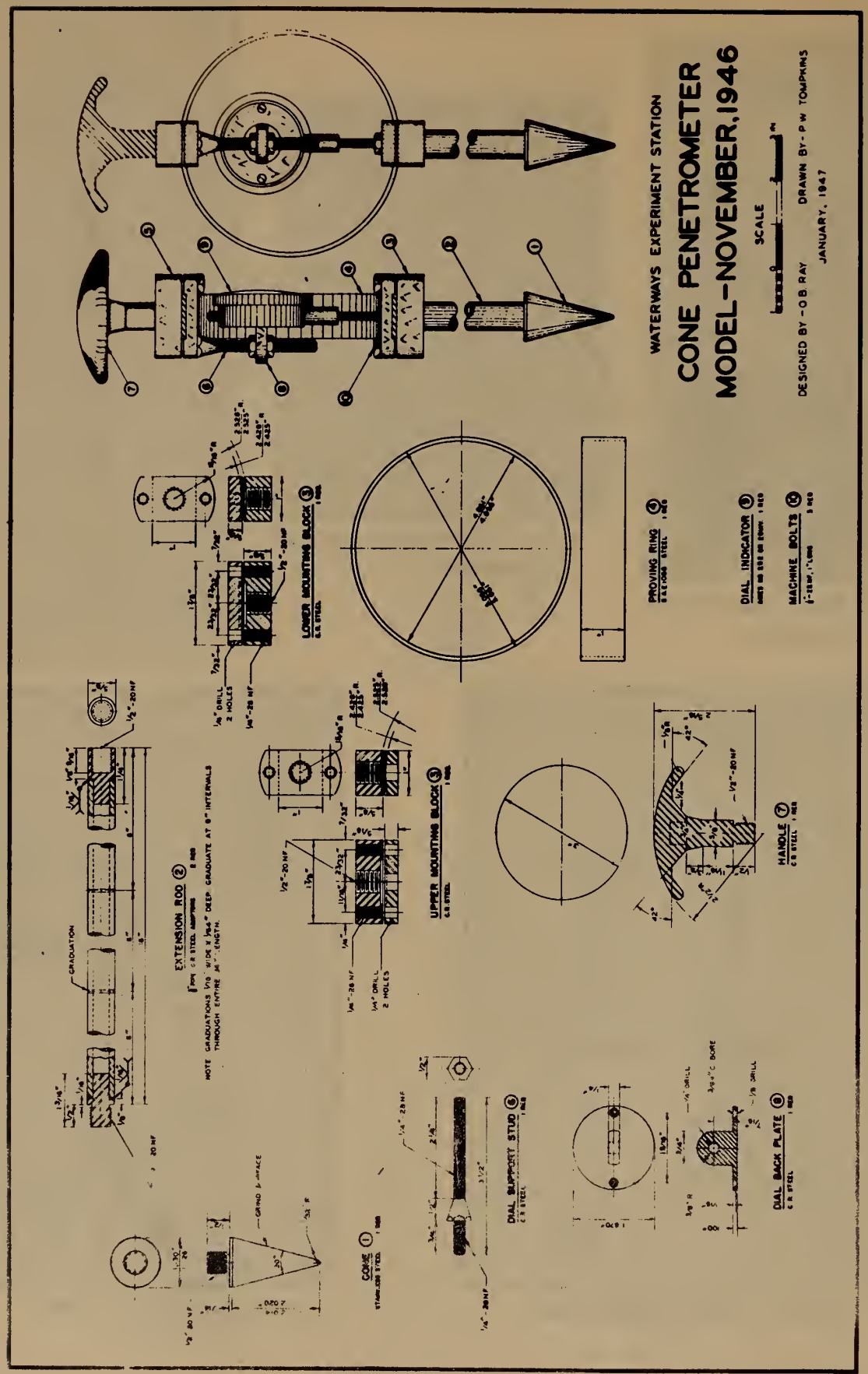

WATERWAYS EXPERIMENT STATION
CONE PENETROMETER
MODEL-NOVEMBER, 1946
SCALE
DESIGNED BY - O.B. RAY
DRAWN BY - P.W. TOMPKINS
JANUARY, 1947
EXTENSION ROD ②
GRADUATION
LOWER MOUNTING BLOCK ③
UPPER MOUNTING BLOCK ④
PROVING RING ⑤
DIAL INDICATOR ⑥
MACHINE BOLTS ⑧
HANDLE ⑦
CONE ①
DIAL SUPPORT STUD ⑨
DIAL BACK PLATE ⑩

FIGURE VC-7

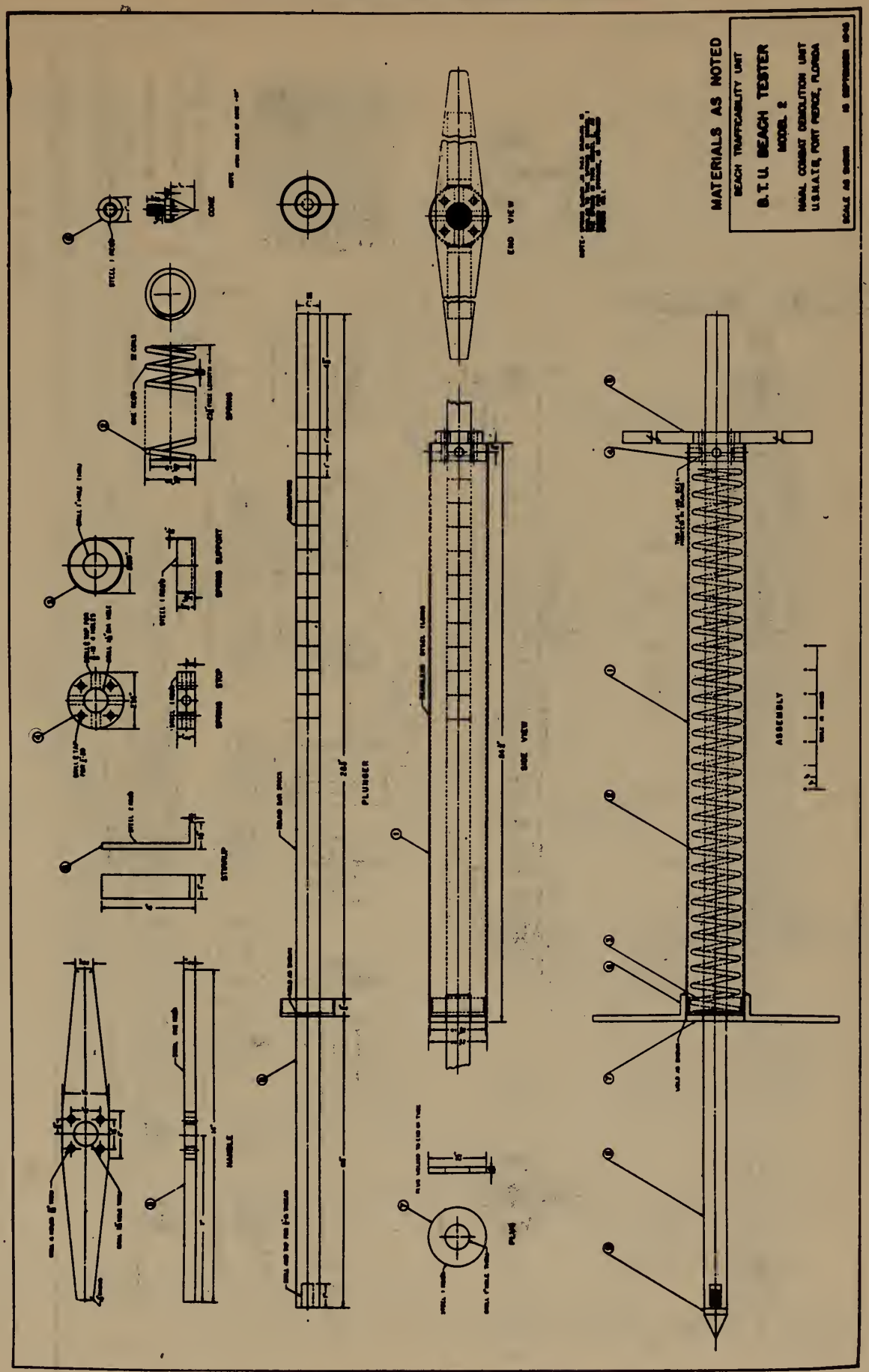

MATERIALS AS NOTED
BEACH TRAFFICABILITY UNIT
B.T.U. BEACH TESTER
MODEL 2
NAVAL COMBAT DEMOLITION UNIT
U.S.N.A.T.S. FORT PIERCE, FLORIDA
END VIEW
SIDE VIEW
ASSEMBLY
PLUNGER
HANDLE
STIRRUP
SPRING STOP
SPRING SUPPORT
CONE
SPRING
PLATE

FIGURE V C-8

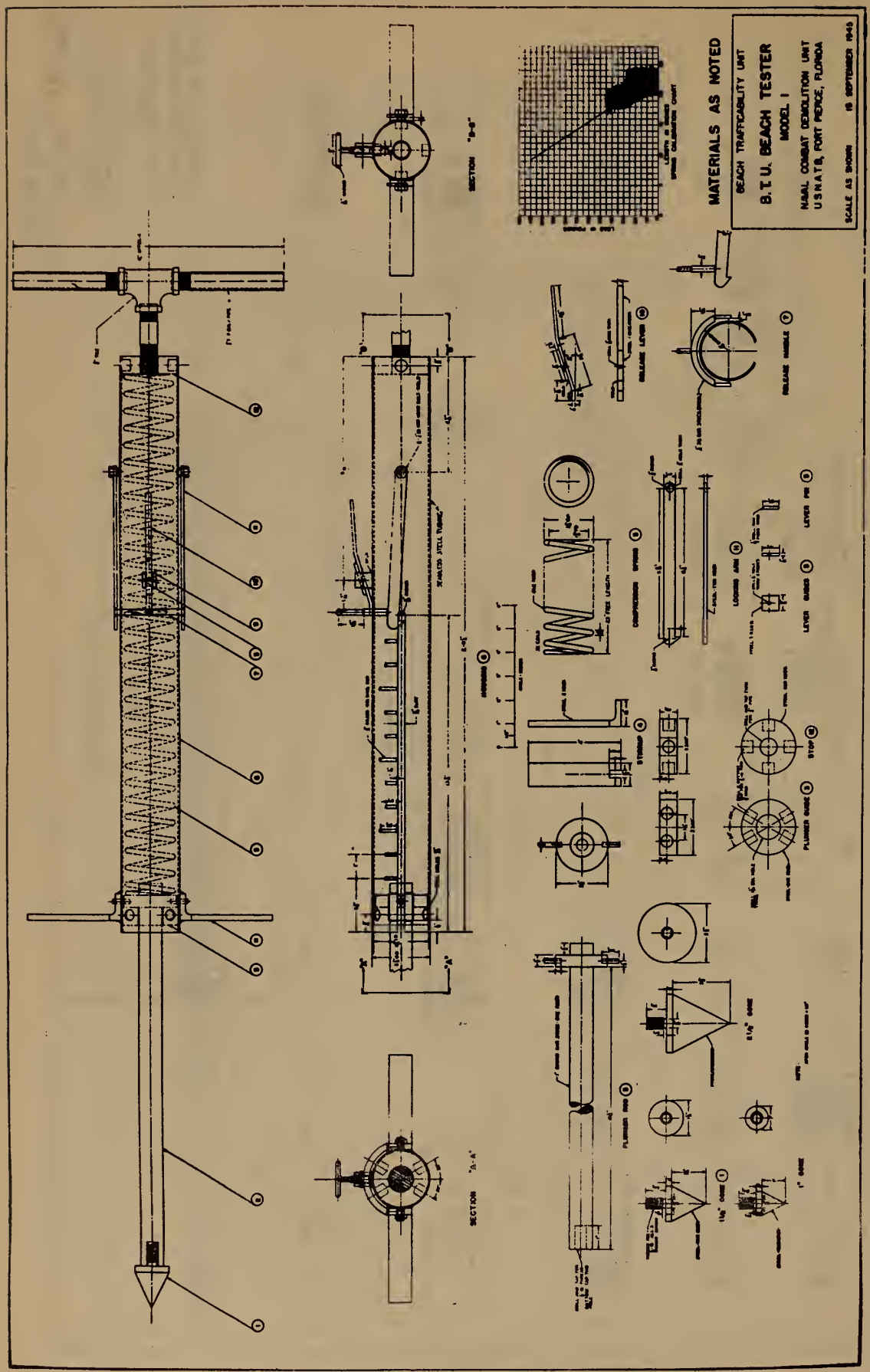

MATERIALS AS NOTED
BEACH TRAFFICABILITY UNIT
B.T.U. BEACH TESTER
MODEL I
NAVAL COMBAT DEMOLITION UNIT
U.S.N.A.T.B. FORT PIERCE, FLORIDA
SCALE AS SHOWN
15 SEPTEMBER 1945

FIGURE V C-9

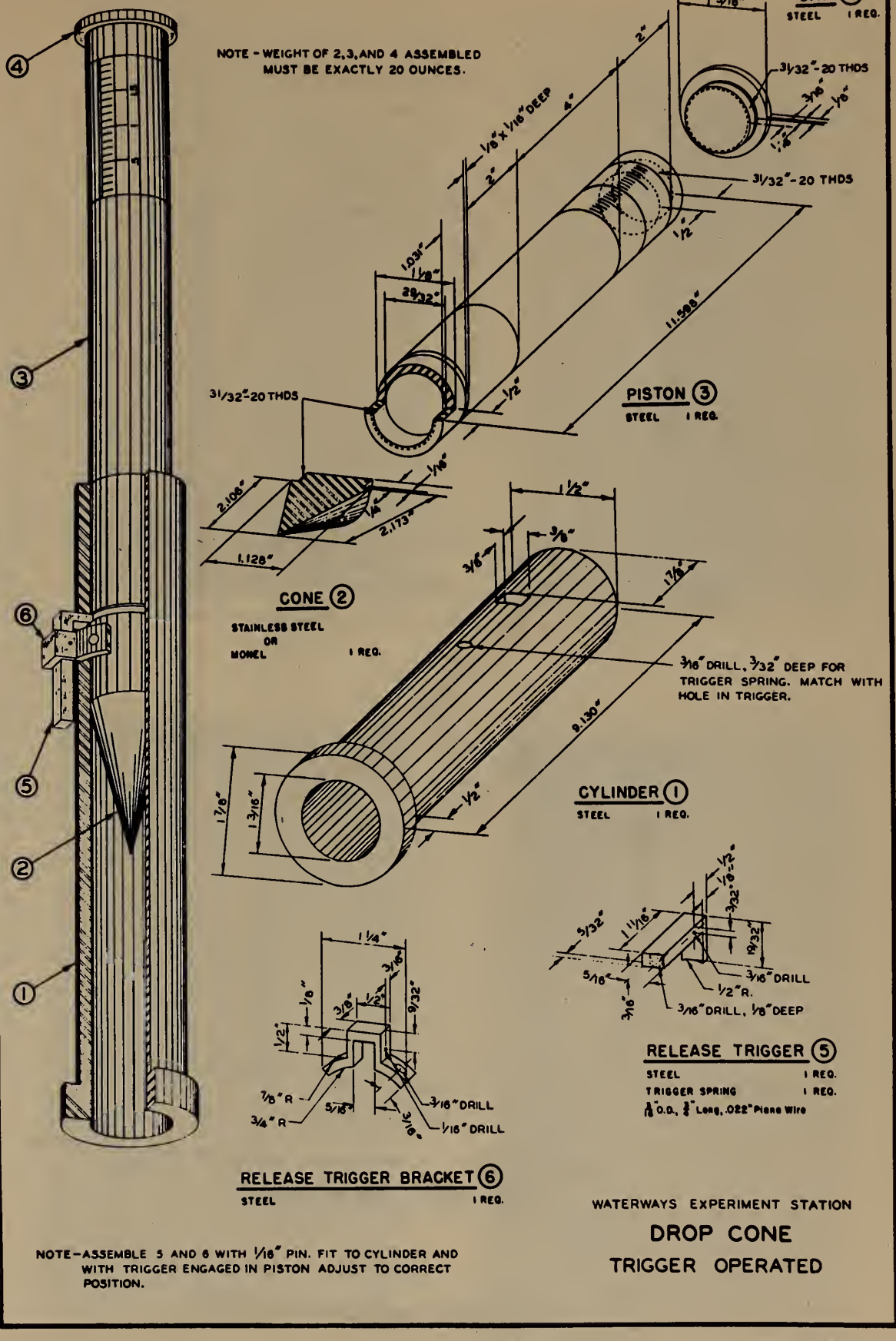

NOTE - WEIGHT OF 2,3, AND 4 ASSEMBLED MUST BE EXACTLY 20 OUNCES.
CAP ④
STEEL 1 REQ.
1 5/16"
3 1/32"-20 THDS
3/16"
1/8"
1/8" DEEP
1/8" x 1/16"
2"
4"
2"
3 1/32"-20 THDS
1/2"
1.039"
1 1/8"
28/32"
11.598"
1/2"
PISTON ③
STEEL 1 REQ.
3 1/32"-20 THDS
1/16"
2.108"
1/4"
2.173"
1.128"
1 1/2"
3/8"
3/16"
1 7/8"
CONE ②
STAINLESS STEEL OR MONEL 1 REQ.
3/16" DRILL, 3/32" DEEP FOR TRIGGER SPRING. MATCH WITH HOLE IN TRIGGER.
9.130"
1 7/8"
1 3/16"
1/2"
CYLINDER ①
STEEL 1 REQ.
1 1/4"
3/16"
1/16"
3/8"
1/2"
9/32"
1/2"
7/8" R
3/4" R
5/16"
11/16"
3/16" DRILL
1/16" DRILL
RELEASE TRIGGER BRACKET ⑥
STEEL 1 REQ.
1/2"
1 1/8"
3/32"
19/32"
5/32"
1 1/16"
5/16"
3/16"
3/16" DRILL
1/2" R.
3/16" DRILL, 1/8" DEEP
RELEASE TRIGGER ⑤
STEEL 1 REQ.
TRIGGER SPRING 1 REQ.
1/8" O.D., 3/4" Long, .022" Piano Wire
NOTE-ASSEMBLE 5 AND 6 WITH 1/16" PIN. FIT TO CYLINDER AND WITH TRIGGER ENGAGED IN PISTON ADJUST TO CORRECT POSITION.
WATERWAYS EXPERIMENT STATION
DROP CONE
TRIGGER OPERATED



FIGURE V C-10

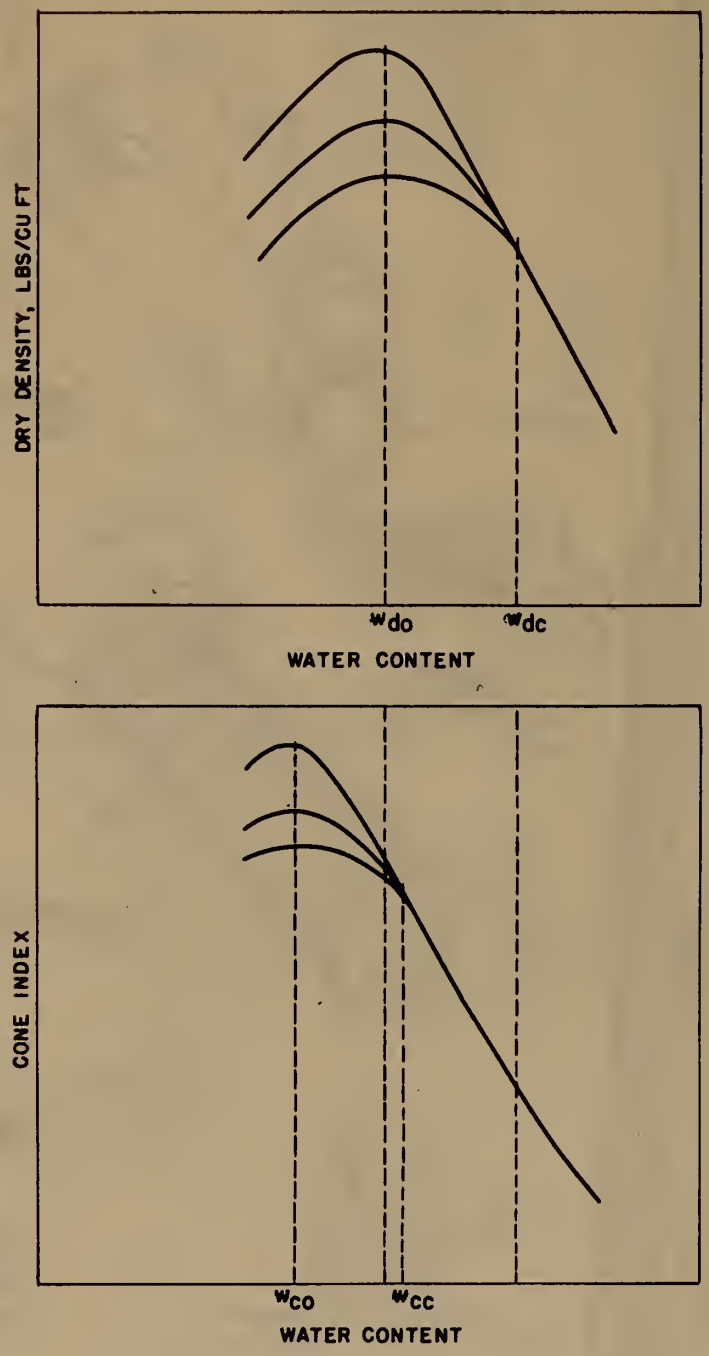



FIGURE VC-11 - MOISTURE, DENSITY AND CONE INDEX RELATIONSHIPS

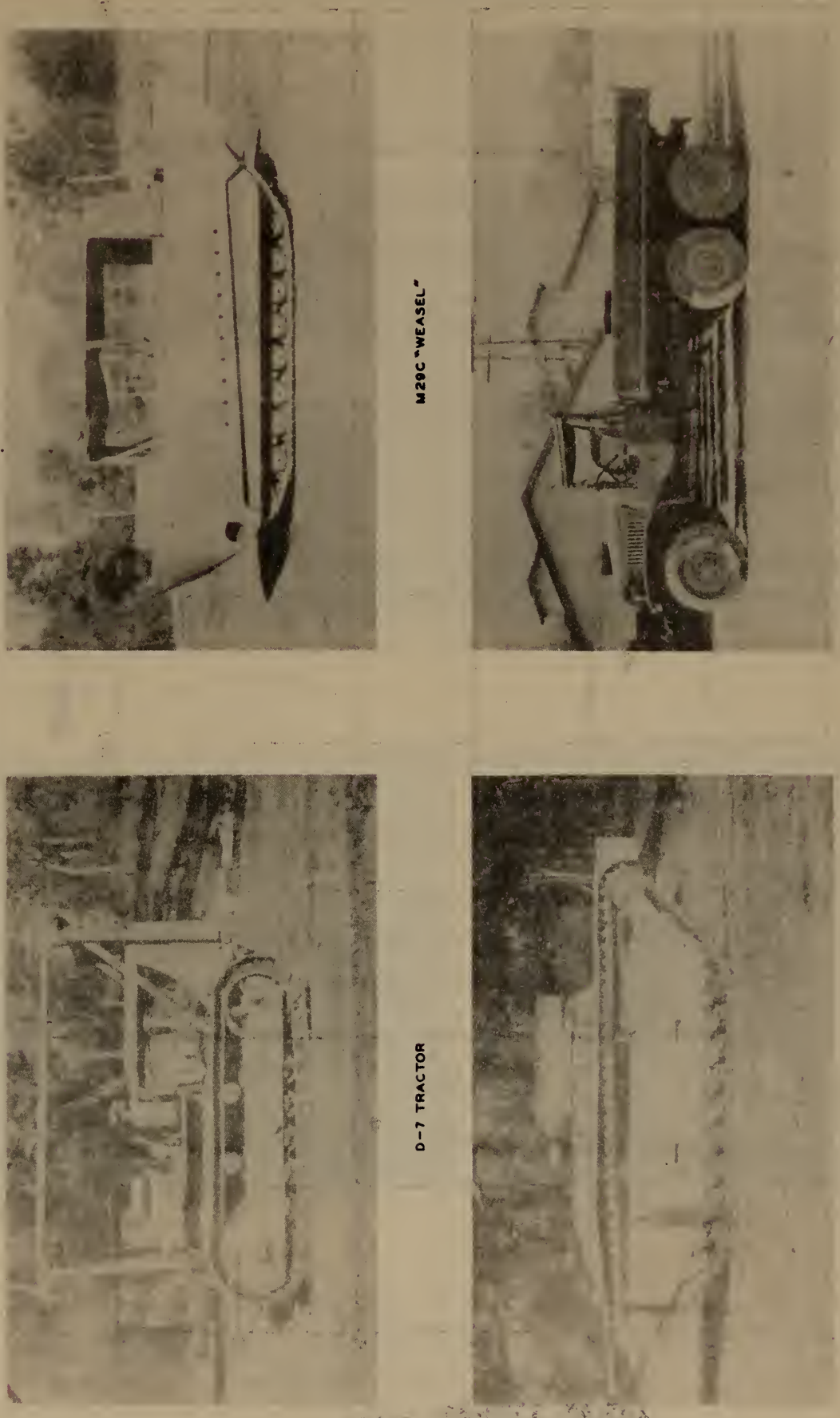



FIGURE V C-12
VEHICLES USED IN TEST

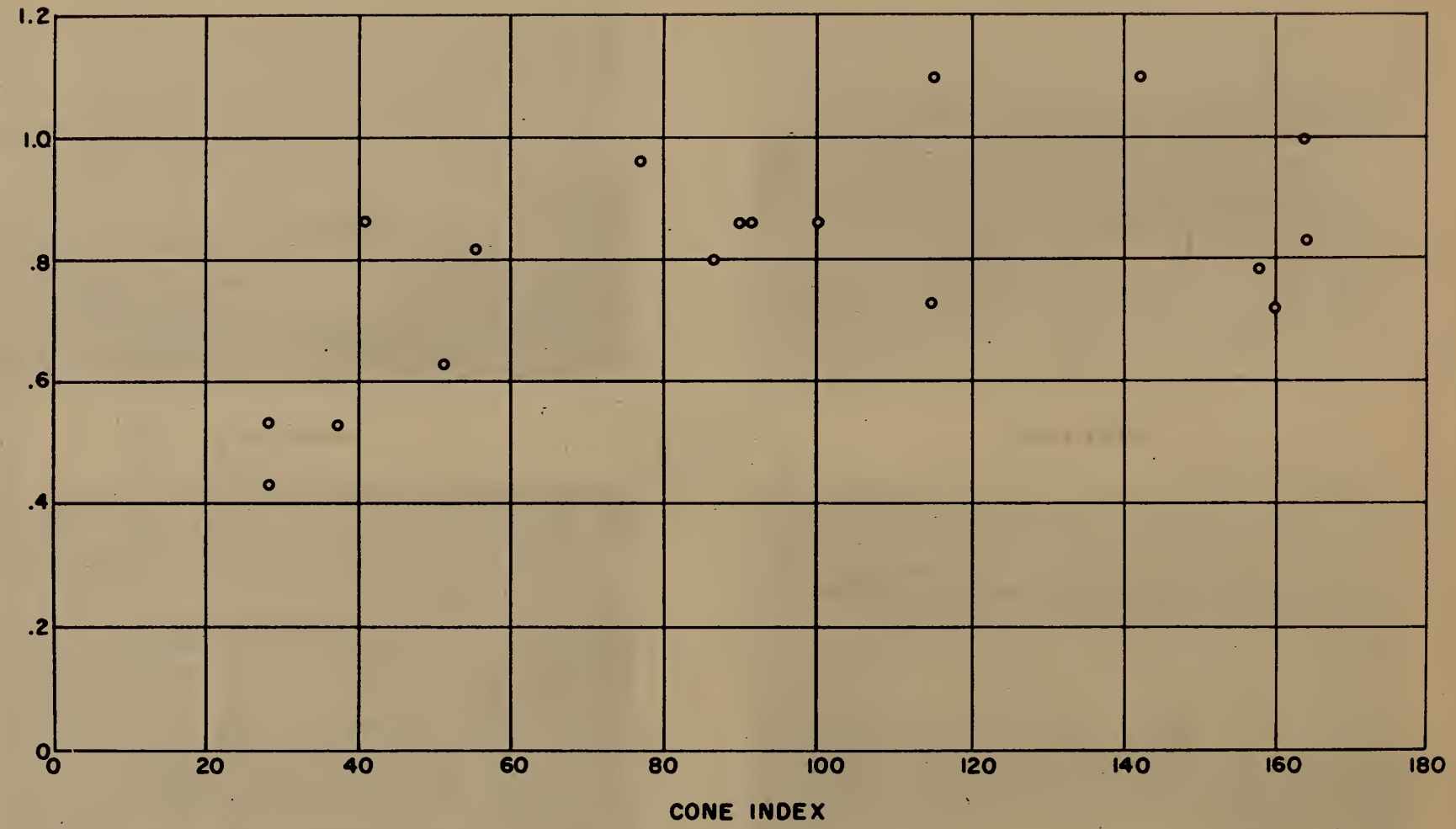



FIGURE VC-13 - CORRELATION OF CONE INDEX
WITH RATIO OF DRAWBAR PULL TO WEIGHT OF VEHICLE
D-7 TRACTOR ON PREPARED CLAYEY SILT



RESTRICTED Security Information



a. DUKW bogged down at edge of surf after standing still for a few minutes; tire pressure was too high.



b. DUKW being pulled out.



c. DUKW tire impression in moist sand.

FIGURE VC-14

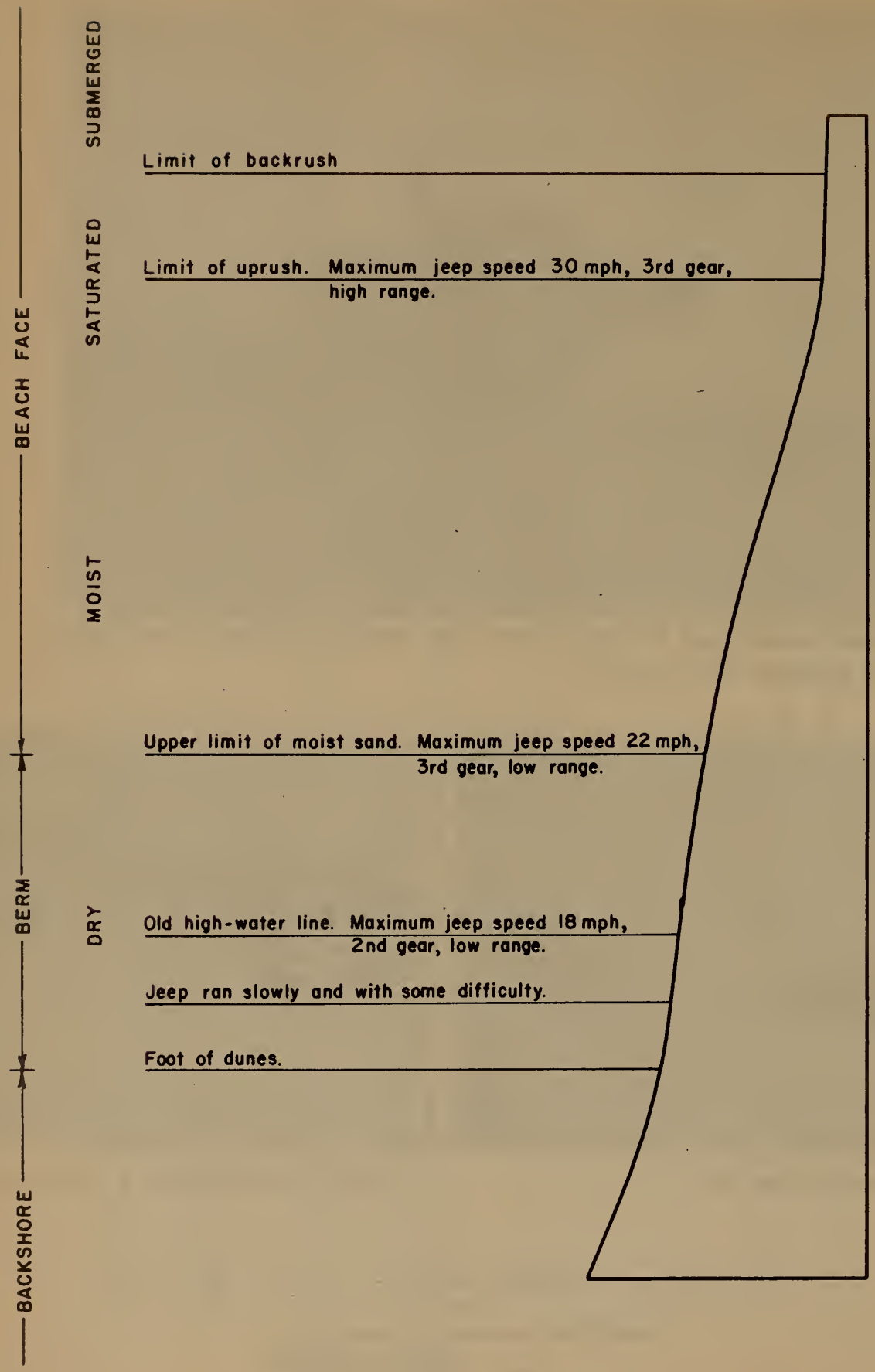



FIGURE VC-15 - PROFILE OF MOSS LANDING SOUTH BEACH SHOWING VEHICLE SPEEDS PARALLEL TO BEACH

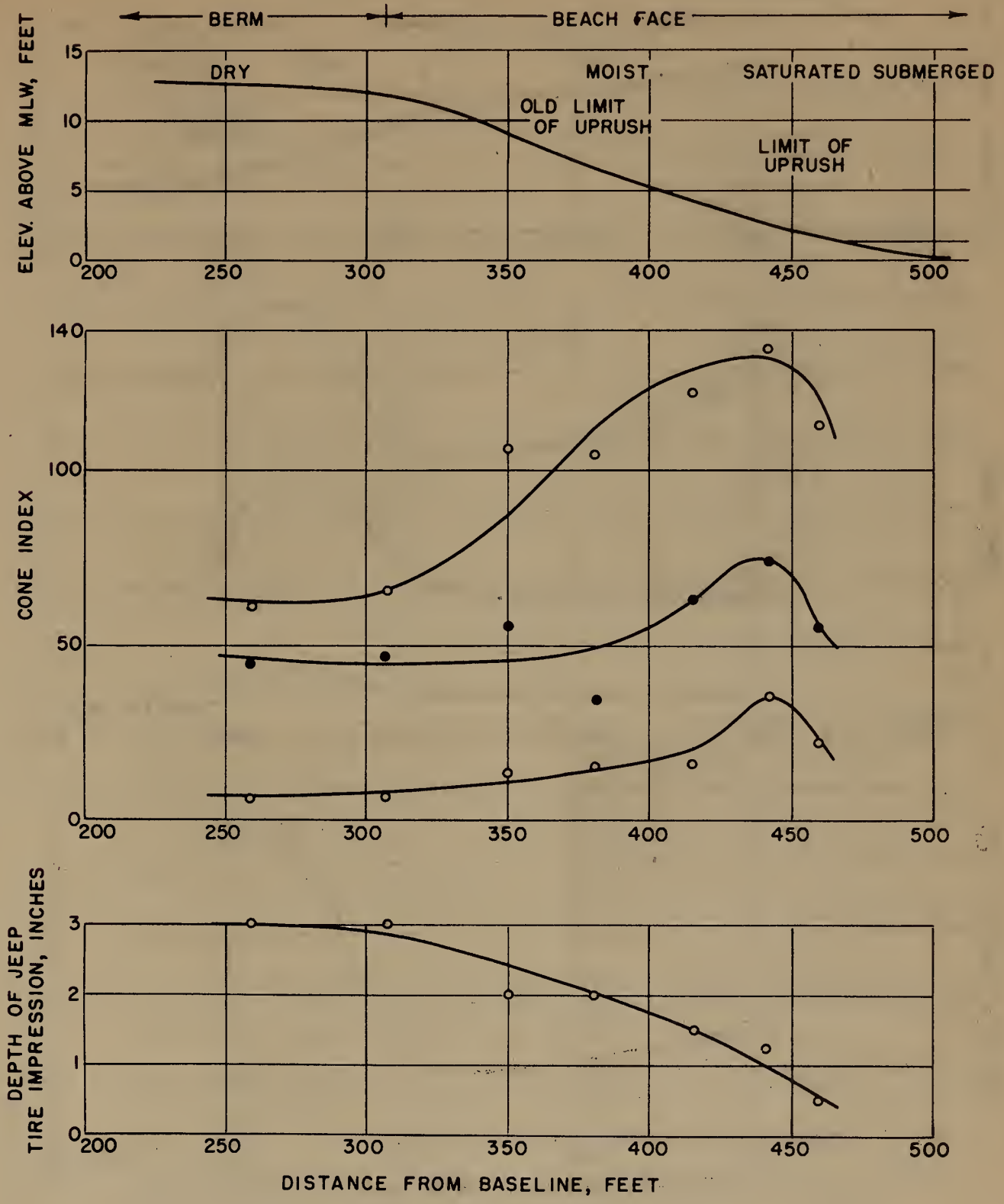



FIGURE VC-16 - VARIATION OF CONE INDEX AND DEPTH OF JEEP TIRE IMPRESSION ACROSS PROFILE OF SANTA MARGARITA R. BEACH

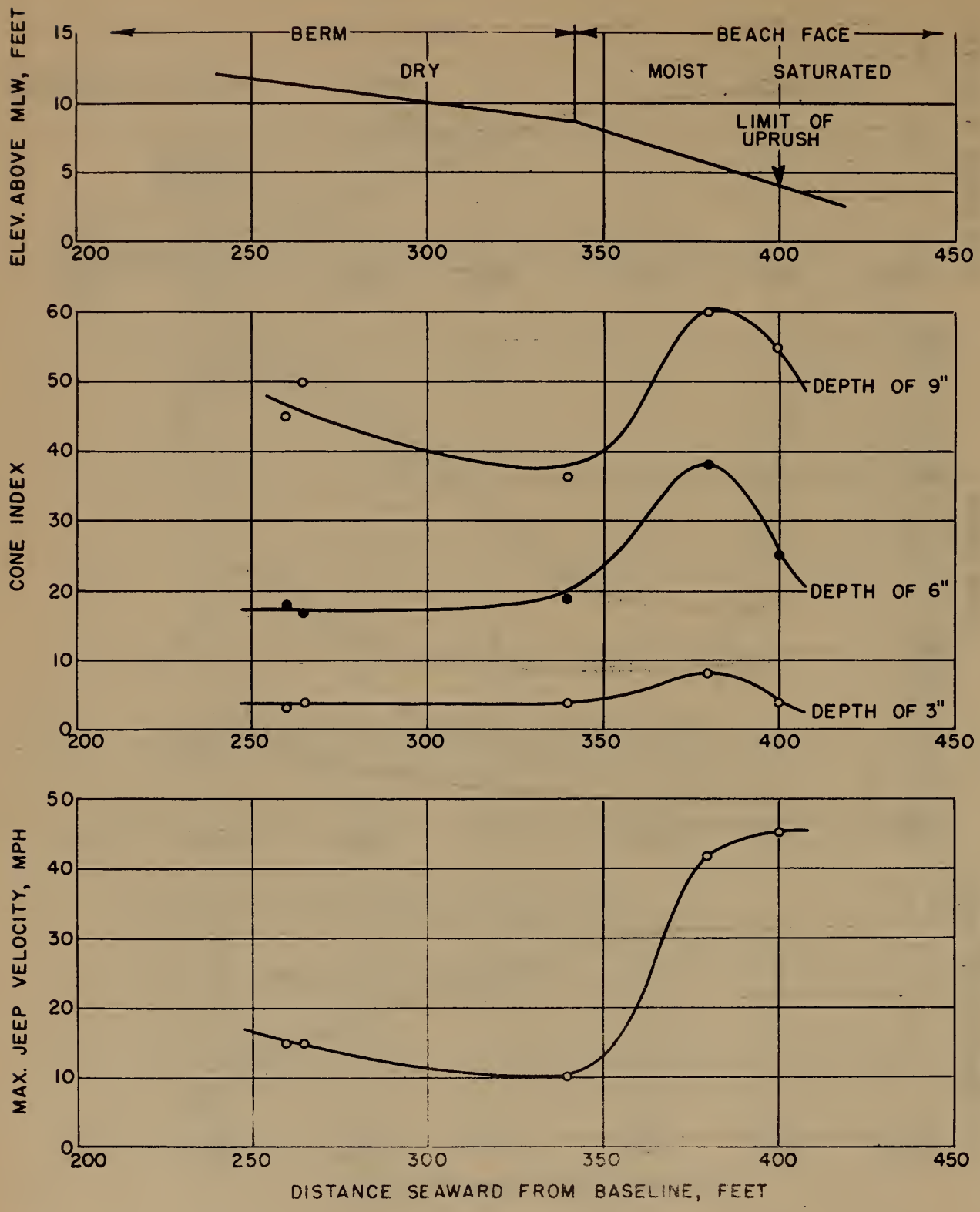



FIGURE V C-17 - DISTRIBUTION OF CONE INDEX AND MAXIMUM JEEP VELOCITY PARALLEL TO BEACH ACROSS PROFILE OF SANTA MARGARITA R. BEACH

# D. GENERAL DISCUSSION OF BEACH SOIL STABILIZATION

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## D. GENERAL DISCUSSION OF BEACH SOIL STABILIZATION

### 1. Introduction

Although some beaches are able to sustain the motion of wheeled and tracked vehicles, in many cases the soil is not sufficiently stable to enable traffic to pass from landing craft to the backshore. For example, while tracked vehicles can operate on most sandy beaches, wheeled vehicles are soon bogged down on all but the flattest beaches. On muddy areas both wheeled and tracked vehicles have great difficulty in operating, as for example, in the landings at Inchon, Korea. Thus, in amphibious warfare, landing operations may be slowed considerably or even entirely prevented due to the nature of the beach material.

The problem is exemplified by the following report of an observer on amphibious training exercises at Little Creek, Va. (VD-5)*. "The landing crafts with track and the Ducks came out of the water on to the beach and advanced to the dune line without difficulty. All other types of equipment except track propelled equipment had serious difficulty in getting from the water line to the dune line without assistance. Trucks, jeeps, equipment of this type could hardly advance 30 ft. beyond the water line before down they would go. In other words, as long as there was moisture in the sand, the equipment could maneuver. As soon as they got beyond the moisture line, the wheels would cut into the sand and assistance would have to be given. This entire operation was considered a normal operation without any set-up to show the difficulty of a beach landing. I saw one bottleneck after another where two or three pieces of rubber-tired equipment would mire down and stop the advance of all other equipment".

When the beach material is non-trafficable or has very limited trafficability, vehicles may be got ashore by means of pontoon causeways, landing mats or other types of temporary roadways. Alternative solutions to the problem of enabling vehicles to operate on beaches are:

(i) The design and use of equipment that can operate on any beach.

(ii) The improvement of the properties of the beach material to render it trafficable for ordinary types of vehicles.

(iii) A combined attack by improving the design of vehicles and improving the stability of beach soils.

The process of improving the properties of soils by physical and/or chemical means in order to increase their stability is termed "soil stabilization". Many methods of stabilizing soils have been proposed and used with varying degrees of success. In order for a stabilization process to be satisfactory for use on beaches during an amphibious operation it must not only cause a considerable increase in soil stability but also be rapid and relatively simple in application and should desirably be cheap in cost.

Although soil stabilization is listed separately above as a means of improving the trafficability of a beach, it cannot be entirely dissociated from the problem of equipment design since

(i) the equipment employed in the stabilization procedures must be able to move freely on the beach

*NOTE: Number in bracket refers to reference listed at end of section.

(ii) improved equipment design will lower the stability requirements of the beach soil and lead to easier or more economical stabilizing procedures.

The question inevitably arises as to what degree of stability is required of a beach material and how extensive should be the stabilized zones. The requirements of the Marine Corps in these respects have been concisely stated. In the initial stages of a landing, a $2\frac{1}{2}$-ton truck should be able to cross the beach carrying a load of about 19,000 pounds. In the later stages heavier vehicles must be put ashore. It is estimated that about 10,000 linear feet of stabilized roadway might be required for a unit landing operation.

A comprehensive report on the stabilization of beach materials was prepared in 1949 by the Soil Consolidation Task Committee of the National Security Industrial Association (VD-5). This report has been used as the basis for the preparation of this section of the manual and the information has been supplemented by a description and discussion of developments in soil stabilization processes and procedures during the past two years.

## 2. Soil Stabilization

Mechanical stability, in its widest sense, may be defined as the ability of a system to withstand applied loads without excessive deformations. While high strength and rigidity are important factors in the establishment of a stable system, other characteristics of the system are also of importance. Where repeated loads are involved, a relatively soft durable material may be more stable than a stronger but more brittle material. This is well-known in the case of metals, where the softer bearing metals are used for machine parts in preference to the stronger but brittle cast iron, and the same is also true for highway surfaces and base courses. Furthermore, the properties of a stable material should not deteriorate with time or with repeated use and stress application.

In soils, excessive deformations are produced by non-elastic movements of the soil under the applied loads. Thus the stability of a soil for trafficability purposes requires that there should be no excessive plastic deformation, that the soil should be durable and that there should be no deterioration of the stability with repeated use. Brittleness is rarely, if ever, a characteristic of non-trafficable soils. Thus, the problem of soil stabilization is mainly that of increasing the strength of the soil without simultaneously producing any deleterious increase in brittleness.

The strength of a saturated clay or mud increases with a reduction in water content. The strength of a sand increases with a reduction in void ratio. Both the decrease in water content of a saturated soil and the decrease in void ratio are equivalent to increasing the closeness with which the soil grains are packed together, that is, increasing the dry density. Densification of a poorly-graded soil may be achieved by the addition of additional soil grains which will fit in the relatively large voids of the original material thus producing a well graded soil. Whether the grain size distribution of a soil is changed or not, densification may be achieved by compaction if the soil is dry or moist or by consolidation if the soil is fully saturated.

The strength of a soil may also be increased by the introduction or addition of a cementing agent which adheres well to the surfaces of the mineral

grains involved and which shows only small variation in cementing power under the temperature and moisture conditions prevailing at its service location.

In important factor in reducing the stability of soils is the presence of water in the voids. If excess water is present, the application of loads may result only in an increase of the neutral stress in the water instead of increasing the effective stresses between the soil grains and thereby the soil strength. Furthermore, the buoyant weight of a soil below the ground water table is only about one-half of its weight above the water table, and frictional strength due to the weight of the soil itself is correspondingly reduced. In the extreme case, the application of load to a loose saturated sand may result in liquification and a complete loss of stability, whereas if the sand were dry or moist, the only effect would be that of compaction. Hence, the elimination of excess moisture from the voids or the prevention of excess moisture accumulation in the voids constitutes a third method of increasing soil stability. Water can be removed from the voids by some form of drainage and it has been proposed that moisture accumulation can be prevented by waterproofing the soil with bituminous or other organic materials.

Thus, three general methods are available for increasing the stability of soils: (i) Densification by compaction, consolidation, improvement of grading or combinations of these methods, (ii) Addition of cementing agents, (iii) Elimination of excess water from voids and prevention of excess moisture accumulation.

A number of procedures for soil stabilization by these methods have been proposed, of which the more important ones are summarized below. The general description of the methods and their application to beach soil stabilization has been so well presented by the Soil Consolidation Task Committee of the National Security Industrial Association that, with minor modifications, the following description is reproduced from their report (VD-5).

## 3. Available Methods of Soil Stabilization

Out of the innumerable theoretical possibilities of stabilizing soils, a limited number of methods and systems have crystallized as practical solutions under normal economic conditions. By their very nature, some of these methods are more advantageous for use with cohesive soils, others for use with non-cohesive soils, or granular soils.

The following stabilization methods are in more or less common use at the present time:

### a. Densification

With respect to greatest potential density and corresponding greatest potential shear resistance, the soils most easily stabilized possess enough coarse component particles to form a connecting internal skeleton, and just enough fine particles to fill the void space even in the presence of free water. In accordance with the findings of Feret (VD-3), this condition usually holds in a system containing 2 parts of coarse to 1 part of fine materials. In accordance with the facts adduced so far, the first thing to be considered in any stabilization project is the possibility of densification. Improvement of a soil by this method will decrease the needed contribution from other methods.

Densification may be attained by either (i) mechanical means or (ii) granulometric improvement. Densification by mechanical means involves the compaction of the soil by some type of equipment. Tractors and vibrating equipment are most satisfactory for cohesionless soils and sheepsfoot rollers for cohesive soils. Improvement of granulometry involves the addition of filler and binder soil to coarse-grained material, or of coarse-grained material to silt and clay soils. Because of the approximate ratio of 2:1 of coarse to fine grained soil necessary, this method is most economical when the coarse-grained material is available on the site.

b. Introduction of Cementing Agents

(1) Soil-cement - (applicable to both cohesive and granular soils). In general, cement requirements increase with increasing clay content; however, relatively large amounts of cement may be required in the case of granular soils possessing large voids ratios, since in such a case the cement serves as both filler and cementing agent.

(2) Soil-lime - (application similar to that of soil-cement, but with smaller ultimate compressive strength and with slower rate of hardening because of different nature of the hardening process.) This method has not been developed to the same extent as soil cement and a good deal of fundamental work remains to be done.

(3) Soil-bitumen - (the bitumen being represented either by asphaltic cutbacks and emulsions or by tars). Depending upon granulometry of the soil and procedure of adding the bitumen, the following variations exist:

(i) oiled earth (surface treatment of earth roads)

(ii) soil-bitumen proper (waterproofing of cohesive soils); bitumen requirements decrease with improving granulometry;

(iii) water-proofed granular stabilization (establishment of optimum granulometry plus waterproofing with bitumen);

(iv) sand-bitumen; the bitumen is added as a cohesion-giving factor, however, the sand must possess a certain amount of natural filler, sufficient to pass the original or modified Florida bearing test, or such filler must be added to improve the granulometry of the sand.

(4) Soil-resin - The development and application of soil-resin is following closely that of soil-bitumen. Resinous materials - natural, modified natural, or synthetic - often possess greater water-proofing power with small percentages of resin (the effectiveness of less than 1% of resin being often greater than that of 7% of bitumen), better adhesiveness to the mineral surfaces than bitumen, and greater cementing strength than bitumen. Accordingly, resins are employed for:

(i) water-proofing of cohesive soils with small amounts - less than 2% on the basis of the dry weight of the soil - of completely or partially neutralized resin and rosin (abietic) acids; and

(ii) water-proofing and cementing of cohesive or non-cohesive fine-grained soils by means of artificial resins or mixtures of artificial and natural resins. The percentage of resin required for this purpose increases with increase in void ratio of the material to be cemented. In other words, resin requirements decrease with improvement in granulometry.

In recent years many resins, both natural and synthetic, have been tried for soil stabilization. The following list of some of these resins, together with brief notes on their effects, is taken from a report by T. W. Lambe (VD-4):

(a) Natural Resins:

(i) Vinsol resin - imparts almost no strength to soil, but appears to waterproof the finer grained soils.

(ii) Rosin - when added to a soil and then reacted with certain metal salts, forms insoluble gels which aid stabilization.

(iii) Resin Stabilizer 321 - a rosin derivative, reduces the rate and amount of water adsorption, and furnishes sodium ions for ion exchange.

(iv) Stabinol - A rosin derivative mixed with Portland cement, waterproofs the coarser soils.

(v) NSP-121 and NSP-252 - materials made by National Southern Products Co., have given some water repellency to certain soils.

(vi) NVX - a product of the Hercules Powder Co., has given some water repellency to certain soils.

(vii) Lignin - a waste from the paper industry, has been used to waterproof and impart a little strength to soil.

(viii) Shellac - set obtained by evaporation of a solution of an ester of shellac with maleic acid or by evaporation of solvent from a solution of shellac in alcohol. Sand treated with shellac can have high strength, but practically all of this strength is destroyed by water immersion.

(ix) Natural Resins from Tropical Areas - a number have been investigated for waterproofing. Manila Copal resin and Wallaba resin showed slight effectiveness.

(b) Synthetic Resins

(i) Aniline-Furfural - waterproofs and imparts some strength to soil.

(ii) Polyvinyl Alcohol (PVA) - forms tough, flexible films

when evaporated from aqueous solutions. Films are water soluble and to date attempts to make them insoluble have been unsuccessful.

(iii) Polyvinyl Acetate - gives very high strength to sand, practically all of which is destroyed by water immersion.

(iv) Resorcinol-formaldehyde - gives some strength to sand, but much of it is lost by water immersion.

(v) Others

(a) Urea-furfural and phenol-furfural resins
(b) Phenol-formaldehyde combinations
(c) Urea-formaldehyde resins
(d) Calcium sulfamate-formaldehyde resins
(e) Ethocel
(f) Methyol ureas and melamine

The most satisfactory method of stabilizing beach sands so far developed is a resinous method of stabilization. This is the aniline-furfural method developed by Dr. Hans F. Winterkorn at Princeton University and is described in further detail in Section VE of this manual.

c. Chemical Stabilization

This term is normally reserved for cases where one, two or more chemicals are added to a soil and form a cementing material by chemical reaction. Well-known examples of this are the use of sodium silicate alone in calcareous, aluminous and ferriferous soils, and the use of this chemical together with Ca-, Fe- or Al- salts or with aluminates or ferrates. For example, sodium silicate mixed into soil and then reacted with calcium chloride forms a gel which fills the voids in the soil and binds the soil grains together. Successful applications of these methods have been mostly in the field of deep foundations. Their application to surface soils is still very much in the development stage.

Calcium chloride and to some extent sodium chloride have also been investigated as soil stabilizing materials. However, these chemicals are not strictly chemical stabilizers since they do not result in the formation of a cementing material. Calcium chloride is deliquescent and sodium chloride has the property of attracting and holding water. Hence, these salts are able to maintain the soil in a moist condition, thereby facilitating compaction and increasing stability by the "apparent cohesion" effects of moisture films.

A satisfactory method of soil stabilization based on the polymerization of calcium acrylate by means of suitable catalysts has been developed at the Massachusetts Institute of Technology. Field investigations are being conducted by the U. S. Corps of Engineers to develop the method into a practical field procedure. The method is further described in Section VE of this manual.

Other materials which have been investigated as soil stabilizers

include:

| | |
|---|---|
| Tung oil | Molases |
| Linseed oil | Mineral oil |
| Cottonseed oil | Sodium carbonate |
| Castor oil | Calcium carbonate |
| Rubber latex | Paraffin |
| Plasticized sulphur | Hydrofluoric acid |
| | Ionic deturgents |

While several of these materials have beneficial stabilizing effects under certain conditions, none can be considered a satisfactory stabilizing material.

d. Fusion or Thermal Stabilization

This method refers to (i) the baking of cohesive soil in situ to make a passable surface or in kilns to be subsequently employed as artificial aggregate for granular stabilization, or (ii) the use of thermite and fluxing agents to fuse the grains of beach sand together, as being investigated under the direction of Mr. Palmer of the Bureau of Yards and Docks.

e. Electro-chemical hardening

This stabilizing process involves the following phenomena;

(i) water-removal by electrosmosis;

(ii) alteration of the surface-chemical properties especially the water-affinity, of cohesive soils and, possibly

(iii) destruction of some of the soil constituents with resultant formation of cementing substances.

f. Water removal

This method may involve either the removal of excess gravity water (by ditching, wellpointing, subsurface drains, etc.), or non-gravity, i.e., capillary and film water, by evaporation, electro-osmotic movement, thermo-osmotic movement, or internal drying.

4. The Problem of Water Removal

In soil engineering, removal of excessive water-drainage - is justifiedly considered one of the most important stabilizing procedures. This holds true even when, and especially when, additional stabilizing procedures are employed. Except for untreated granular soils, excess moisture signifies low bearing power. As far as the excessive moisture is non-capillary gravitational water, it can be removed by normal drainage methods.

The really objectionable water, that which has the greatest effect on the bearing power of a soil system is non-gravitational capillary and film water which cannot be removed by ordinary drainage methods. These types of water occur in relatively fine-grained soils having a coefficient of permeability less than about $10^{-3}$ cm/sec. The procedures employed at the present time to remove these types of water are based on the following fundamental

phenomena:

(i) evaporation due to a difference in vapor pressures of the liquid and the surrounding atmosphere, respectively;

(ii) electro-osmotic movement of water in a soil system to which an electric potential has been applied;

(iii) thermo-osmotic movement of moisture in a soil system to which a thermal gradient has been applied;

(iv) internal drying by introduction of highly water-adsorbing materials.

(v) Consolidation induced by atmospheric pressure

It deserves emphasis at this point, that in order to achieve a certain purpose (i.e., establishment of equilibrium or increase of entropy) nature employs all means at its disposition. For this reason, there is, in actuality, never a so-called "pure" method of water removal; there are only those in which one of the listed phenomena (either evaporation, or electro-osmosis, or thermo-osmosis, respectively), predominates, with the two others playing a secondary though still important role.

a. Evaporation

Every liquid (or solution) possesses a vapor pressure which is characteristic for the liquid (or solution) and is also a function of the temperature; the higher the temperature, the higher is the vapor pressure. Evaporation occurs if the partial pressure (of the molecules of which the liquid is composed) in the surrounding atmosphere is smaller than the vapor pressure of the liquid.

The speed with which evaporation occurs is a function of the difference in vapor pressure, but also depends on whether the evaporating liquid molecules are carried away from the liquid surface only by the slow process of diffusion or by the much faster one of convection. For this reason, anything which induces convection and turbulence (such as stirring and air movement) increases the over-all speed of evaporation.

To evaporate one gram of water at 30°C requires 609 gram-calories, at 100°C., 539 gram-calories; this amount of heat has to be supplied from an outside source if the temperature of the system is to stay the same, or it has to be supplied by the heat content of the water involving lowering of its temperature.

If the evaporation from the surface of a water saturated system is faster than supply of water to the surface by capillarity or any other mechanism, then drying will proceed from the surface to the interior of the body; if evaporation equals supply, then a steady flow condition comes into existence with the moisture content of the system remaining constant; if supply exceeds evaporation, water accumulation takes place. Each one of these conditions may occur in natural and stabilized soils.

b. Different Methods of Supplying Heat Energy

Heat energy for the purpose of raising the temperature of a water-solid

system may be applied in a number of different ways. The most important of these are:

(i) direct contact with a body possessing a higher temperature than the water-solid system, resulting in heat exchange;

(ii) irradiation, especially by infra-red light; this light is the more effective the closer the frequencies employed coincide with the characteristic frequencies of the water molecules and the more easily this light is transmitted, i.e., the smaller is its absorption by the non-aqueous or solid portion of the system;

(iii) so-called "electronic" heating in which the material is used as a dielectric in a condensor connected to a source of high-frequency current. "The work performed by the electric field in displacing the electric charges in the system is converted into heat as the charges resume their equilibrium condition." More details concerning the theory and application of this method can be found in an article by Doucerain from the Bulletin de la Societe Francaise des Electriciens, 6, No. 62, Nov. 1946;

(iv) resistance heating, where the system is submitted to the flow of a d.c. electric current. (Such heating results not only in an increase of the temperature of the system, but in case of a moist porous system also in concomitant electro-osmotic phenomena.)

Except for very specific cases the newer methods of supplying heat energy for water removal can not as yet take the place of natural evaporation due to irradiation of solar energy. However, there exists two relatively new methods of removing water which appear to be of a greater though still limited potential applicability.

These are the electrosmotic and thermosmotic method. They are both still in the development stage.

c. Electro-osmotic Movement of Water

This process is described by Terzaghi and Peck (VD-6) as follows: "If two electrodes are driven into a saturated soil and an electric current is made to flow from one to the other, the water contained in the soil migrates from the positive electrode (anode) toward the negative one (cathode). If the cathode consists of a wellpoint the water seeps into it whence it can be removed by pumping.

"The movement of water is due to the fact that the surface of the soil particles carries a negative charge. As a consequence positively charged ions in the water are attracted toward the soil particles and the film of water adjacent to the soil is positively charged because of the preponderance of positive ions. Although there is no sharp boundary between the positively charged and the neutral water, for present purposes the seat of the electric charges may be regarded as a well-defined layer known as the electric double layer. The positive ions, which are concentrated in the water near the soil

particles, are attracted by the negative electrode and repelled by the positive one. Hence, the positive layers together with the enclosed column of neutral water migrate towards the cathode."

This process has been used on several occasions for draining soils prior to excavation but experience with the method has not yet been extensive enough to permit general statements concerning its applicability and economy.

d. Internal Drying

An interesting possibility for removal of excessive amounts of water lies in the water-absorbing and binding properties of a number of materials. It is known from concrete practice that some very light and porous aggregates may absorb as much water as 25% by weight, i.e., this much water has to be added to the dry material in addition to the amount required for the chosen water-cement ratio. The most effective absorbent of this type is probably diatomaceous earth which absorbs water up to 500% of its weight. However, if used for this purpose, the diatomaceous earth must be calcined to remove the diatom fat which otherwise renders the material difficult to wet.

Such internal water absorption, or better, water use, is also a property of all hydraulic cements; a water-cement ratio of 5 corresponds to about 40% of water on the basis of the dry weight of the cement, while plaster of Paris requires about 100% of water to make a good working paste. It should be mentioned here that Portland cement has already been used in practice for such internal drying purposes with the additional benefit of ameliorating the properties of cohesive soils in mud flats.

e. Consolidation by Atmospheric Pressure

This method has been used for stabilizing the banks of deep excavations in inorganic silts and organic clays. Wellpoints surrounded by pervious material are sunk into the ground and a seal is created near the ground surface. A vacuum is then developed in the wellpoints. Before the vacuum is produced both the upper surface of the fine-grained soil and the soil surrounding the wellpoint are acted on by the atmospheric pressure of about 1 ton/sq. ft. After the vacuum has been produced the pressure on the soil around the wellpoints is practically zero while that on the surface of the soil is still equal to about 1 ton/sq. ft. Under this pressure gradient, water is gradually squeezed out of the soil into the wellpoints until the effective pressure in the soil adjoining the row of wellpoints has increased by an amount equal to the atmospheric pressure.

5. Soil Characteristics Affecting the Stabilization of Beach Sands.

On sandy beaches, the soil consists of relatively coarse (diameter greater than about 20 microns), relatively rounded grains containing in their voids variable amounts of moisture and air. On the berm the sand is practically dry, on the beach face it is moist and the water content increases gradually as the sea is approached, the sand becoming saturated near the limit of uprush. The water in the voids has the same average composition as sea water.

Because of the classifying effect of wave movement, the sand particles are of relatively uniform size, although the average size varies with different beaches. An approximate correlation has been found to exist between grain size and beach face slope for a number of beaches on the Pacific Coast of the United

States and on the Hawaiian Islands and the coefficient of uniformity for the sand on these beaches varied only between the low values of 1.3 and 3.0.

The average sea water contains about 4.3% of dissolved material, over half of which is common salt. Sea water is not only saline but also slightly aklaline. The latter fact is of importance with respect to the rate of reaction of a number of stabilizing mixtures.

Thus, the following features are typical of beach sands:

(i) sand grains of rather uniform size and relatively rounded

(ii) variable water content

(iii) salinity and alkalinity of water

These characteristics of the soil have important effects on the stabilization of beach sands.

6. Consequences of the Uniform Size of Sand

An aggregation of spherical particles of uniform size may have different densities depending on the manner in which the particles are arranged. The loosest orderly arrangement is one in which the particles are stacked one vertically above the other, each being in contact with six others. In this condition, the void ratio of the mass (ratio of volume of voids to volume of solids) may readily be shown to be 0.92. The densest possible packing is represented by the traditional pyramid of cannon balls. In this arrangement the void ratio is 0.35.

The difference in void ratio of spherical particles arranged in their loosest and densest conditions indicates the importance of densification procedures since the mechanical stability increases as the void ratio decreases. Expressed in terms of density for spherical particles having a specific gravity of 2.65, a void ratio of 0.92 corresponds to a dry density of 86.6 lb/cu. ft. and a void ratio of 0.35 corresponds to a dry density of 122 lb/cu. ft. Alternatively, changing the void ratio from 0.92 to 0.35 would reduce the thickness of a 1-ft. layer of the particles by about $3\frac{1}{2}$ inches or 30%.

While actual sands show variations in shape, surface roughness, size and type of packing, all of which influence their site density and their potential maximum density, a number of these factors cancel each other out and therefore the above calculations give a good indication of the order of magnitude of the effects of densification of natural sand deposits.

Because of the uniform size of the grains and the absence of finer material to fill the voids between the grains, beach sands have higher void ratios than well-graded materials. In consequence of these higher void ratios, beach sands require the addition of from 7% to 20% or more of material passing a No. 200 sieve to make them susceptible to bituminous stabilization or to stabilization by means of economical amounts of Portland cement.

7. Densification of Cohesionless Material by Vibration

For the purpose of densifying natural deposits of cohesionless materials,

vibration has been found to be most effective. The vibration effects, on the beach face, of breakers and foam lines during a high tide are no doubt partly responsible for the sand in this section of the beach being more compact and more trafficable than that of the dry berm.

The Soil Consolidation Task Committee report (VD-5) states that a vibratory densification method should produce:

(i) a decrease in the friction between the particles by giving them sufficient kinetic energy to keep them at an average distance from each other which is larger than the peaks of their surface roughness; also we expect it to

(ii) give the particles enough amplitude of vibratory movement (free path) so that they are able to slip over their neighbors; to

(iii) have a vertical downward force component, either gravitational or induced by surcharge to help keep the particles in their lowest possible location once this is gained; and to allow

(iv) enough time between reversal of direction of kinetic movement to permit a particle to jump to a location of lower potential energy, i.e., of greater entropy.

The described action mechanism indicates the dominant importance of the amplitude of the actual particle movement and the much lesser importance of the frequency as long as it stays below a certain critical value.

The results of a comprehensive investigation of the compaction of cohesionless sands by vibrations caused by plates on the surface of the sand conducted by the California Institute of Technology (VD-1), were published in March 1950. The object of this research program was to provide criteria for the design of vibration compacting equipment and for the evaluation of existing equipment. The report states:

"From the data obtained during this investigation, the following conclusions can be drawn regarding the compaction of sand by vibration. The limitations of the tests should be kept in mind in weighing the conclusions. These limitations include the facts that the sand was well graded and contained less than two percent of material finer than the 200 mesh sieve, the sand was in a pit with walls and bottom of a different type of soil, and both static and dynamic unit loads were low. In addition, it is important to note that dynamic forces were kept low relative to the dead weight, so that impact conditions would not develop.

(i) Maximum compaction is obtained when the vibrator is operated at the resonant frequency of the vibrator-sand mass system.

(ii) For a given dead load and vibrator base plate, ultimate vibrator settlement increases approximately in direct proportion to an increase in dynamic force.

(iii) Dead load, as long as it is greater than dynamic force, is relatively unimportant in producing settlement.

(iv) Within the range of moisture contents used in these tests, the lower

the moisture content of the sand the greater the degree of compaction which can be produced by vibration.

(v) Operating at constant frequency and dynamic force, vibrator settlement is proportional to the logarithm of the time of vibration.

"The following conclusions can be drawn regarding factors which affect the resonant frequency of the vibrator-sand mass system:

(i) An increase in dynamic force produces a decrease in the resonant frequency.

(ii) An increase in vibrator dead load produces a decrease in the resonant frequency.

(iii) An increase in base plate diameter produces a decrease in the resonant frequency.

(iv) An increase in the average initial dry weight of the sand in the pit (i.e., a decrease in initial void ratio) produces an increase in the resonant frequency."

For plates having diameters between about 15 and 50 inches and weights between about 500 and 1200 lb. the following empirical equation was developed for predicting the resonant frequency of a vibrator-sand mass system:

$$f = \frac{\sqrt{g}}{2\pi} \sqrt{840 \frac{\rho}{G} \left(1.64 - \frac{F}{W}\right) + 0.55 \frac{GR}{W}}$$

where $g$ = acceleration of gravity, inches per second per second

$\rho$ = initial unit weight of sand, pounds per cubic foot

$G$ = shearing modulus of elasticity of the sand, pounds per square inch

$W$ = vibrator dead load, pounds

$F$ = dynamic force, $\pm$ pounds

$R$ = base plate radius, inches

Resonant frequencies computed from this formula were found to agree well with values observed in the field testing program.

8. Menisci Effects in Moist Sands

The presence of water in the voids of a sand has two effects. First, it has a lubricating effect causing a slight reduction in friction between the sand grains, and second, it results in the formation of water menisci at the points of contact between the grains. The surface tension in the menisci causes the grains to be pulled together and results in the development of frictional resistance to deformation. The frictional strength which is due to the menisci is not due to any visibly applied force, and the sand behaves as if it possessed cohesive strength. For this reason, the strength due to the menisci is called

"apparent cohesion." The apparent cohesion is immediately lost if the water menisci are destroyed.

While the decrease in friction due to the lubricating effects of the water is slight, the apparent cohesion is appreciable, and a moist sand has greater resistance to deformation than the same sand in a dry condition. This is partly responsible for the fact that the moist section of a sandy beach is much more trafficable than the dry section.

The problem of calculating the magnitude of the apparent cohesion in moist sands has been attacked by a number of research workers. The basic picture of the analytical approach represents two spheres touching each other and being connected by an annular concave water wedge. If the angle described by the radius of the particle to the centre of the water-ring and by the radius to the outside edge of the water-ring is called $\beta$, R. A. Fisher (VD-2) developed a relationship between the tensile stress needed for rupture, the surface tension of water, the radius of particles and the angle $\beta$ for sand in loose and dense conditions. According to this relationship, the apparent cohesion, for a given arrangement of the sand grains, is a function of the wetting wedge angle (i.e. of the water content) and the diameter of the sand particles.

If it is assumed that the maximum values of $\beta$ are developed and that the surface tension of water is about 70 dynes per cm., then the following values are obtained for the apparent cohesion of aggregations of uniform spheres in open and close-packed conditions:

| Particle Size in mm. | Cohesion in $gm/cm^2$ | | Cohesion in p.s.i. | |
|---|---|---|---|---|
| | Open Packing | Closed Packing | Open Packing | Closed Packing |
| 2 | 0.485 | 0.68 | 0.0069 | 0.0097 |
| 0.2 | 4.85 | 6.8 | 0.069 | 0.097 |
| 0.02 | 48.5 | 68.0 | 0.69 | 0.97 |

Although these are relatively small forces, their practical significance with respect to the trafficability of beaches is important. The moist section of a sandy beach has much better trafficability than the dry berm and while it is not known how much of this improvement is due to increased compaction, the apparent cohesion undoubtedly makes an important contribution. Furthermore, the improved trafficability of the dry berm after it has been moistened by a heavy rain is significant. Thus it appears possible that after proper densification, the cohesion which must be induced in the surface layers of a sand by stabilization treatment in order to produce adequate trafficability may be quite small, of the order of several p.s.i., instead of the large increases in cohesion which have been striven for in the past.

The good trafficability of moist fine sand also suggests that irrigation of the dry portion of beaches consisting of such sand should be considered as a potential stabilization method.

9. The Influence of Salinity and Alkalinity of Beach Sands

A number of stabilization methods are definitely affected by the salinity

and/or alkalinity of the soil material. It is important to have actual data on the magnitude of this influence on different types of stabilizers. It must be kept in mind that excessively salty beaches may be brought to the concentration level of sea water by washing it with the latter. Also, sea water is often the only water available for construction. Therefore, its influence, if employed for stabilization, must be ascertained.

10. The Variability of Beaches

Because the majority of beaches in the United States and in many other parts of the world are relatively flat and are composed of sandy (predominantly quartz) materials, these conditions are normally envisaged as characteristic of the typical or usual type of beach. However, beach materials and conditions vary widely. This subject has been so well presented in a memorandum by Dr. Girard Wheeler to the chairman of the Soil Consolidation Task Committee of the N.S.I.A. that a section of the memorandum is quoted below (VD-5):

"The geological prognostication is that many leading beaches in a future war will be 'unusual'. One need only remember the many and complex factors involved in forming beaches to appreciate how varied beach types may be. Some variables in beach formation are wave and current action, direction and magnitude of long-shore drift, the fetch of open water, depth of the off-shore zone, character of the shore-line (coastal plain rectilinear or bay headland indented), the nature of bottom sediments just off-shore, and of the sediments provided by adjacent rivers or by marine erosion of nearby headlands.

"Under the wartime exigencies of rapid planning, last-minute changes of plans, 'island-hopping' schedules determined on the spur of the moment, and quick shifts of the political scene, the selection of beach-heads is often made on uncomfortably short notice. Peace-time investigations of beach consolidation should, therefore, anticipate atypical conditions and avoid overgeneralization of the problem.

"For landing operations, then, a 'sandy-beach' must be understood to mean any shore zone of non-cohesive soil, in-shore from the low-water line, and comprising not only the emerged, wetted area exposed at low tide, but also the upper beach above normal high water. In some cases it will include also the beach ridge terrain still farther inshore (e.g. Cape Canaveral), or sand dune areas (e.g. Rehobeth Beach, Jones Beach), etc.

"The shore zone can be steeply or gently sloped, or a combination of steep and gentle slopes. At low water the wetted area may be narrow or wide. Groundwater (and the saturated capillary fringe above it) may be relatively close to the beach surface for a broad width of shore zone, or it may be reached at progressively deeper depths as one moves inshore from the high-water line. The soil materials of the beach may range in size between very fine sand (with traces of silt and clay) to coarse sands and gravels (up to the size of cobbles and boulders found on shingle beaches). The sands may be well graded or poorly graded, may consist predominantly of quartz, or carbonate, or acidic or basic volcanic debris.

"A thorough review of beach-head problems met during the last war would, very probably, establish the importance of fairly detailed investigation of beach types in all possible military zones, with special reference to geological properties. As regards beach sand compositon, for example, it is well known that many of the Pacific beaches, far from being quartzose, consist

almost entirely of carbonates (the coral islands, atolls, etc.). At the same time, many islands in the Pacific are composed chiefly of volcanic materials of a basic (basaltic) nature, and free quartz is conspicuous by its absence. The great beach on the east coast of Honshu, Kujukuri-hama, which was to have been used as one of our major landing beaches in the invasion of Japan, is very high in magetite and ferro-magnesian minerals. On the other hand, volcanic beach sediments in the Atlantic and Mediterranean would often run high in quartz. Such variability in composition should certainly be of consequence in thermal, chemical and electrical stabilization methods.

"Not only composition, bur size and shape of sand grains may be of importance. The landing of Iwo Jima, it is said, was handicapped, among other things, by the poor tractibility of the fluffy upper beach soils, composed entirely of volcanic ejectamenta.

"Off New Guinea, during the island-hopping campaign of the Southwest Pacific, there was a fiasco subsequent to a landing on a (supposedly) dry beach for which high and low water relationships had been very imperfectly visualized. Incidents such as this are bound to happen during any war, but they serve to emphasize the fact that during a war intelligence is often unable to keep up with operations. Naturally, this places a premium upon the thorough coverage of whatever inter-war planning may be done."

In addition to the variability of sandy beaches described by Dr. Wheeler, landings may have to be made on beaches composed of fine-grained soils such as silts and clays. When these types of soils have a relatively high water content, they become soft and plastic and are termed "muds". Mud flats present serious problems of trafficability and their characteristics are very different from those of sandy beaches.

Thus, while stabilization processes are especially required for typical sandy beaches, it is also desirable that processes should be developed for all types of soils and conditions.

11. Summary of General Discussion of Beach-soil Stabilization

The preceding discussion of beach soil stabilization may be summarized as follows:

(i) The trafficability of a beach may be improved by stabilization of the beach soil.

(ii) Stabilization of a soil may be achieved by densification of the soil, by the introduction of a cementing agent to increase the strength of the soil, or by removal of water from the soil.

(iii) Densification may be attained either by improvement of the grading of the soil grains or by compaction.

(iv) Cementing agents may include such materials as cement, lime, bitumens, resins, or combination of chemicals.

(v) Water removal may be effected by gravity drainage, pumping, thermal methods causing evaporation, consolidation, internal drying or electro-osmosis.

(vi) Vibration is the best method of compacting beach sands.

(vii) Apparent cohesion in moist sands makes an important contribution to the stability of the sand.

(viii) Cementing agents for stabilizing beach soils should not be affected by the salinity or alkalinity of the soil material.

(ix) Beach materials vary widely in type and in composition. While the majority of beaches are sandy, landings may have to be made on beaches composed of gravels, cobbles, boulders or muds.

(x) Beach-soil stabilization processes are required for all types of granular soils and muds.

It is clearly improbable that a single soil stabilization process will prove most suitable for all types of beach soils and conditions. In any given case, the selection of the stabilization procedure to be used will depend on:

(i) the time available for stabilizing the soil;

(ii) the amount of ship space available to transport the stabilizing material and equipment;

(iii) the local availability of stabilizing materials;

(iv) specific beach properties.

12. References

V D-1: California Institute of Technology - "An Investigation of the Compaction of Soil by Vibration".

V D-2: Fisher, R. A. - "On the Capillary Forces in an Ideal Soil; Correction of Formulae Given by W. B. Haines" - The Journal of Agricultural Sciences, Vol. 16, Part 3, July 1926, pp. 492-505.

V D-3: Joisel, A. - "Ou en est la Granulometre du Beton" - La Technique Moderne Construction, Vol. 3, No. 3, June 1938, Paris, France.

V D-4: Lambe, T. W. - "Stabilization of Soils with Calcium Acrylate" - Journal, Boston Society of Civil Engineers, April 1951.

V D-5: Soil Consolidation Task Committee of the National Security Industrial Association - "Soil Stabilization" - September 1949.

V D-6: Terzaghi, K. and Peck, R. B. - "Soil Mechanics in Engineering Practice" - John Wiley and Sons, Inc., New York, 1948.

E. DEVELOPMENT OF PRACTICAL METHODS OF BEACH-SOIL STABILIZATION

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## E. DEVELOPMENT OF PRACTICAL METHODS OF BEACH-SOIL STABILIZATION

### 1. Introduction

In the spring of 1947, the Bureau of Yards and Docks arranged for a survey and analysis to be made of the theory and practice of soil stabilization. This survey was used as the basis for subsequent investigations of beach sand stabilization. The following aspects of soil stabilization were reviewed:

(i) Granular Soil Stabilization

(ii) Soil-cement

(iii) Soil-bitumen

(iv) Soil stabilization by means of resin

(v) Soil stabilizing effectiveness of one-fourth neutralized abietic acid.

(vi) Soil stabilizing effectiveness of synthetic resins

(vii) Recent patents in soil stabilization

(viii) General principles and practice of soil stabilization

(ix) Results of war-time research in soil stabilization with new types of chemicals.

(x) Injections

(xi) Electronic heating

(xii) Electrosmotic phenomena

(xiii) Thermosmotic phenomena

As a result of this survey it was concluded that for earliest practical results, investigations should be limited to a small number of chemicals which had shown greatest promise in the work of several agencies. The following chemicals were suggested for investigation:

(i) sodium silicate - sodium aluminate (indicated by work at the Armour Research Institute sponsored by the Civil Aeronautics Administration).

(ii) vinyl alcohol derivatives employed together with water-repellents (indicated by work sponsored by the U. S. Engineer Board).

(iii) melamin resins (U. S. Engineer Board).

(iv) aniline-furfural resins (indicated by work sponsored by the Civil Aeronautics Administration at the University of Missouri and at Princeton University, respectively).

It was proposed that laboratory investigations should be made of the effectiveness of these materials and that simultaneous field investigations should be conducted to develop the best method of applying the stabilizing materials on sandy beaches.

As a result of these recommendations, a research project, sponsored by the Bureau of Yards and Docks, was started at Princeton University in November 1947 under the direction of Dr. Hans F. Winterkorn. The results of this research were presented in monthly progress reports and a final report was published in July 1950 (VE-6 and 7). Since that time, the project has been continued at the Winterkorn Research Institute, Princeton; monthly reports have been issued. A detailed manual on soil stabilization was in preparation at the time this section was written.

In conjunction with the work at Princeton, field investigations have been made at Port Hueneme and an investigation of the thermite-fusion method of soil stabilization has been initiated and directed by Mr. L. A. Palmer of the Bureau of Yards and Docks.

In the earlier field tests at Port Hueneme, the thermite-fusion method and stabilization with Portland cement, sodium silicate and aniline-furfural were investigated. The results were reported by Mr. G. E. Sandford (VE-4). The cement-treated sections passed all the traffic tests. However, beach sands require the addition of relatively high proportions of cement in order to form a stable material. The thermite method was effective and provided a good roadway, but it was felt that the method needed further investigation in an effort to eliminate some of its undesirable features.

The setting and hardening of the aniline-furfural proceeded relatively slowly. Ten days after application it had not hardened sufficiently to provide a good roadway, but it later produced a high strength. However, the spreading and mixing procedures were relatively crude and, considering all factors, it was concluded that treatment with aniline-furfural should be further investigated since it required less material and labor and since it can be applied more easily and faster.

Based on the results obtained in these tests, Mr. Sanford made the following recommendations:

(i) "That investigation of the thermite method be continued in an effort to reduce the quantity of material and to devise a method of application which will reduce the time and labor now required."

(ii) "That investigation of the aniline-furfural method be continued with special emphasis on shortening the time required for curing."

In accordance with these suggestions and the results of laboratory investigations, the research work at Princeton was concentrated mainly on the development of the aniline-furfural method of beach sand stabilization with special emphasis on the testing of catalytic and other admixtures which would hasten curing and hardening.

An independent soil solidification project, sponsored by the Engineer Research and Development Laboratories of Fort Belvoir, Virginia, has also been conducted at the Massachusetts Institute of Technology. After testing many materials in search of a chemical which can solidify soils, the best solidifier so far developed is calcium acrylate, used in conjunction with a suitable catalyst. Field tests on soil stabilized with calcium acrylate have been conducted at Fort Belvoir.

A review of the development and investigations of the aniline-furfural and thermite-fusion methods of beach sand stabilization and the clacium acrylate method of soil solidification is presented in the following pages.

2. The Aniline-Furfural Method of Beach Sand Stabilization

The aniline-furfural method is a special form of resinous stabilization of soils, the resulting material being called a sand-resin. The principles of resinous stabilization have been excellently described by Dr. Winterkorn in a monthly report (VE-5) dated 1 October, 1949. "The basic principle involved in resinous beach stabilization is the formation in the sand of a synthetic resin, or of a mixture of natural and synthetic resins, with or without an active or inert filler, by the reaction of two liquids separately added to the sand and subsequently mixed intimately and compacted, the latter preferably by vibration. The resin or resin mixture should set and solidify sufficiently fast to allow use of the treated section by traffic within 24 hours after construction, with only such surface impression by the traffic as is normally ironed out by the traffic itself. In sand systems of this type actual rupture occurs normally only after deformation of more than 1/8 inch.

"In order to assure the optimum properties of the synthetic resin binder and its setting and solidifying within the specified time, the two (or sometimes three) liquids in solution or dispersion must be applied in the exact predetermined ratios. These ratios are the most important single factor governing the success of the method; second in importance is the absolute amount of materials applied to the sand. This requirement of exact maintenance of a predetermined liquid - liquid ratio renders it necessary to comply strictly with the specifications covering the liquid pumping and spraying systems.

"So far the most successful methods of resinous sand stabilization are based on the ability of two molecules of aniline to react with one molecule of furfural, producing a synthetic resin which has very strong adhesion to the sand minerals and which has good cohesive properties. The rate of reaction can be and usually is controlled by means of positive or negative catalysts, while the properties of the resin produced can be controlled by the admixtures of plasticizers and/or of inactive or active filler materials.

"There are two classes of aniline - furfural stabilization methods, viz. (i) where these liquid materials are applied substantially pure with only the required catalysts and/or plasticizers dissolved in one or both liquids; and (ii) where the liquids as applied represent cutbacks (solutions or dispersions) of pitches, waxes, distillation residues of petroleum, and other similar materials, in aniline and furfural respectively (in a similar way as asphaltic cutbacks are formed by solution of asphalt in kerosene and in naphtha), and where these cutbacks are applied in such relative proportions that the liquids acting as solvents react completely with each other forming a synthetic resin which is compatible with the material carried originally in solution and which forms in combination with this material a strong and durable resinous binder. Of course, between these extreme types (i) and (ii) there exists a multitude of possible intermediate types, which, however, are normally classed with either (i) or (ii).

"In both laboratory and field tests, the halogen salts of the trivalent

Fe- and Al- ions have proved to be the most important catalytic and pH - control agents in the resinification of analine-furfural. The only other material having comparable effects was pentachlorophenol. After testing beach sands from all parts of the world, Dr. Winterkorn arrived at the following general conclusions (VE-6):

(i) "The influence of the mineralogical and chemical character of beach sands on the susceptibility to stabilization by means of the aniline-furfural method can be controlled by the type and amount of catalyst added. This fact makes the method one that can be applied universally.

(ii) "The amount of stabilizing agent required increases with increase in specific surface area and total pore volume of the sand system under consideration. Practical guides for the evaluation of these properties are the fineness modulus, the uniformity coefficient, and observation on the presence or absence of porous grains such as pumice and fragments of exorkeleta possessing large internal surfaces.

(iii) "Stabilizer requirements range from about 3% by weight of the dry sand for solid grains of intermediate fineness and uniformity to 7.5% or even 10% for sands possessing large specific surfaces such as those derived from shell and coral."

Test results indicate that there is a critical value of stabilizer content at which there is a marked change in strength of the stabilized soil. For stabilizer contents above this critical value the strengths were often of a different order of magnitude than for stabilizer contents below this value.

The amounts of $FeCl_3$ and pentachlorophenol normally required to produce the required catalytic effects on sands can be dissolved or dispersed in aniline. However, these admixtures increase the viscosity of the aniline to values comparable with those of cut-back asphalts. This means that they can be handled at normal temperatures with normal bituminous spreading and mixing machinery.

In order to decrease the brittleness and increase the plasticity of the stabilized soil, a number of field and laboratory tests have been conducted in which pitch and other materials have been added to the aniline-furfural - catalyst stabilizer. The results indicate that the substitution of pitch for aniline and furfural to some extent may give some improvement in the physical characteristics of the stabilized soil.

Considerable difficulty has been encountered in developing satisfactory equipment for applying the aniline-furfural stabilizer in the field. In the earlier field tests (VE-2) stabilized strips were constructed using a train of equipment consisting of a D-8 tractor, spray bars, a Seaman Pulvi-Mixer (a rotary tiller) and a vibrating sled for compaction. A unit carrying the resin-forming materials and an air compressor was drawn alongside to supply the spray bars and the vibrator respectively by means of flexible hose. The Pulvi-Mixer is shown in Figure VE-1, the position of the spray bars in Figure VE-2, and the stabilizing train in Figure VE-3. The proportioning and mixing were not performed satisfactorily by this equipment resulting in variation of the design proportions of the chemicals. The lack of uniformity of the stabilized soil is shown in Figure VE-4. Such a variation affects the setting and hardening time of the resin and causes a weakening of the stabilized soil. However, in spite of these difficulties, all of the strips eventually developed

sufficient strength to withstand the traffic of light vehicles and in some cases, that of trucks with heavy loads.

In order to improve the proportioning and mixing, tests were conducted using a P and H Single Pass Stabilizer, shown in Figure VE-5. This machine is equipped with two 250-gallon tanks, two 120 g.p.h. pumps, two spray bars and a mixer having one set of digging teeth and three sets of mixing teeth. As in the earlier tests, a vibrating sled was used for compaction. Although the mixing and compaction operations were carried out well with this equipment, only one strip of stabilized soil was constructed because of failure of the fluid dispensing system. This failure was due to excessive foaming caused by circulation of the fluids at very high rates.

In this test a 150-foot strip of sand was stabilized with 4% of aniline-furfural. Pentachlorophenol and aluminum chloride were used to accelerate the rate of hardening of the resin, the binder being composed of 80.1% of aniline and furfural (in the ratio of 2:1), 18.7% of pentachlorophenol and 1% of aluminum chloride. Two hours after construction the strip was able to support a slow-moving jeep. An hour later a truck having a gross weight of 18,040 pounds traversed the strip and left no visible deformation. Twenty-four hours after construction the strip could sustain the traffic of a truck weighing about 14 tons.

The cost of the materials used in the stabilization of this strip was about 34 cents per square foot. Neglecting shipping and labor costs, this is about ten times greater than the cost of comparable stabilization using a high-early strength portland cement. However, about 8% of cement would be required compared with the 4% of aniline-furfural.

In spite of the good results obtained in this test, the P and H Single Pass Stabilizer did not prove satisfactory for the continuous stabilization of beach sands. In addition to the failure of the fluid dispensing system, the unit is very heavy and its pumping capacity is too large. As a result of the tests, it was concluded that further improvement in equipment would be necessary before the use of aniline-furfural could be considered a practical and reliable method of stabilizing beach sands. It was also recommended that equipment should be developed to meet the following specifications:

(i) Provided with a positive proportioning system which would dispense two liquids with uniformity.

(ii) Of such design as would facilitate the determination of optimum amounts, speeds and other similar factors.

(iii) Capable of producing intimate mixing (at two or more depths, if possible).

(iv) Capable of producing good compaction

(v) Constructed so that the unit, or train of units, could be transported with relative ease.

The only commercial equipment which might meet these specifications is the Model 54 Roadmixer of the Wood Manufacturing Company. Further tests using this equipment are described by W. A. Mueller (VE-3) as follows: "One of these units was modified for more accurate metering and pumping of the two chemicals.

A screw-type spreader and vibrating compactor were added at the rear of the horizontal-drum pug mill. The complete unit was mounted on Athey Tracks and towed by a D-8 tractor.

"Power for the pug mill was taken from the rear power take-off of the tractor through a drive shaft. Power for the pumps, screw spreader, and vibrating compactor came from auxiliary engines mounted on the roadmixer itself. Another tractor towed tanks for the chemicals alongside the roadmixer with connecting hose lines.

"Traveling at a fixed speed of 17 feet per minute, the machine scooped up beach sand and passed it back through the pug mill. Here it was thoroughly mixed with the correct amounts of chemicals. As the mixed materials emerged from the rear of the pug mill, they were spread and compacted into a strip of any width up to about 12 feet. Depth of initial sand pickup and finished strip was limited only by the power of the tractor. Figures VE-6,7 and 8 show the Wood Roadmixer as used at Port Hueneme. Eight hours after stabilization one of the strips easily supported the 38,600-pound load shown in Figure VE-9.

"A further modification of the Model 54 mixer did away with the auxiliary tractor-drawn chemical tanks. With the mounting of a 1,000-gallon elevated tank on the mixer chassis, the first completely self-contained equipment for aniline-furfural stabilization came into existence. This improved model is shown in Figure VE-10.

"After the California trials, the mixer was shipped to the east coast for service evaluation by Naval Mobile Construction Battalion No. 1 at Little Creek, Virginia. However, before delivery, the mixer was taken to the Engineer Research and Development Laboratory of the Army Corps of Engineers at Fort Belvoir, Va. Operating for the first time in cohesive soil, the aniline-furfural technique and Wood Roadmixer were used to stabilize several 50-foot strips of low-bearing-value clay. Because of the cohesive nature of the soil, the vibrating compactor was replaced with a smooth-wheel roller.

"A full evaluation of the Belvoir tests is not yet available. However, it is interesting to note that the clay strips at Belvoir supported a wheel load of 50,000 pounds several days after stabilization. This load was applied by the center wheels of a special loading cart of the Corps of Engineers."

A strip of stabilized soil constructed with the Wood Roadmixer is shown in Figure VE-11. It is interesting to note that although this method was developed for beach-sand stabilization, it has been successfully used for the stabilization of sandy clays. In addition to the field tests at Fort Belvoir, laboratory tests on the stabilization of Fort Belvoir sandy clay, using aniline-furfural, have been conducted by Dr. Winterkorn. A number of tests were made with the sandy clay at water contents as high as 50% (which is considerably above the liquid limit) in order to test the stabilizer under the worst possible conditions. With the soil at these high water contents, the stabilizing effect of the aniline-furfural (and catalyst) was slowed down, though effective stabilization was attained after several days. However, effective stabilization would be obtained within one day when the water content of the sandy clay was 35%. At a water content of 13.5% and using 10% of aniline-furfural, a compression strength as high as 1200 p.s.i. was attained after one day and lower, while perfectly satisfactory one-day strengths was obtained using only 5% of aniline-furfural with water contents up to 25%.

In addition to these successful tests with sandy clay, aniline-furfural

has also proved satisfactory for the stabilization of highly calcareous sands from Guam, Puerto Rico and the Virgin Islands. However, for this type of material, higher proportions of stabilizer are required.

Since it has been found possible to substitute pitch to some extent for aniline and furfural in the stabilizer, and pitch is both cheaper and more readily available than aniline and furfural, more recent laboratory investigations have been directed to the development of aniline-furfural-pitch stabilizers. For this purpose, the resin-forming liquids consist of solutions of pitch in aniline and pitch in furfural, together with small proportions of suitable catalysts. It has been established that about 5% of this stabilizer (when a high melting point pitch is used and the aniline and furfural constitute roughly 50% of the respective solutions) would normally stabilize beach sands satisfactorily within two hours to meet the needs of emergency operations. It remains to be determined whether the aniline and furfural requirements can be further reduced by the use of low melting point pitches.

The addition of pitch increases the viscosity of the resin-forming liquids and one of the factors limiting the pitch content is the requirement that the viscosity must not exceed that required for satisfactory construction operations. The rate at which the stabilized soil acquires sufficient working consistency is dependent on the melting point of the pitch used in the pitch-aniline and pitch-furfural solutions. The higher the melting point of the pitch the faster is an adequate consistency level attained. However, in general, pitches with high melting points require greater proportions of the more expensive solvents to form solutions having viscosities sufficiently low to be easily applied. Thus the problem is to determine the type of pitch or pitches which require the lowest possible amounts of aniline and furfural to form solutions of the desired fluidity and which combine to form a cementing material that hardens at a sufficient rate for emergency stabilization operations. Work is in progress to develop the best combinations of pitch, aniline and furfural for use in different climatic zones.

In pitch-aniline-furfural stabilization, about 5 to 7% of aniline hydrochloride (dissolved in the aniline of the pitch-aniline solution) is used as a catalyst to speed up the rate of setting and hardening of the cementing material.

An essential feature of a stabilization process to be used during emergency landing operations is that the necessary materials should be readily available in sufficient quantities in times of war. In this connection, Dr. Winterkorn has stated (VE-7): "With respect to the availability of materials required for the present form of the aniline-furfural method, the following situation exists. Production capacity for furfural is adequate and can easily be increased if this should become necessary; the same general situation appears to exist with respect to the nitric acid required for the conversion of benzene to nitrobenzene which by reduction yields aniline, and with respect to hydrochloric acid for the making of the aniline hydrochloride.

"A peculiar situation exists with respect to the benzene from which aniline is manufactured. At the present time most of the benzene is obtained in the coking of coal. An alternate industrial method is available consisting in catalytic conversion of plentiful natural gas. There exists a certain differential favorable to the old method in the cost of the two processes which so far has held back the development of the catalytic process. However, the benzene

situation has been tight for quite a while and even for peace-time purposes it is only a matter of time before the new source must be tapped. Already, a black market in benzene seems to have started with prices almost twice those of the normal market and considerably above those at which synthetic benzene could be produced and sold.

"Under these conditions production of snythetic benzene becomes a necessity and should be encouraged by the Government, especially also in view of the large benzene requirements of the synthetic rubber industry which can be expected to expand in time of war. Proper expansion of the synthetic benzene industry will result in a plentiful supply of this chemical and of its derivatives.

"Considering all the factors involved, the aniline-furfural-pitch method can and should stand as a primary beach stabilization process and provisions should be made that the required materials be available for its use. For these reasons the beach stabilization research conducted during the past year was concerned primarily with making this process as universally applicable and as practical as possible. However, attention was also paid to the possiblities of other methods and materials.

"Reasons for a search for alternate methods are of varied kind and weight. There is always the possibility that a better method or a more economical one may be found. Also, materials required for one method may be short in supply in time of an emergency. Furthermore, certain materials not commonly employed in construction of traffic ways may be available and usable for military purposes in a theater far distant from the normal supply center. This was the case during the second world war with respect to tung oil in China and to natural resins in the Philippines, Malaya, Australia and North Africa.

"For military purposes it is best to have one definite universally useful method and to make certain of the availability of the necessary materials by increasing the production capacity. On the other hand, it is well to investigate the possibilities of materials which might be available in potential war theaters, to find out the best way of using them and to teach commanding officers the theory and practice of such use."

Pitch-aniline and pitch-furfural combinations are actually only typical representatives of a large group of bituminous waterproofing and cementing agents, a thorough exploration of which will take many years. In addition, there are many soil stabilizing materials which have been investigated in the laboratory but which have not yet been tested in the field. Investigations are continuously in progress to develop methods which are more satisfactory or more economical than pitch-aniline-furfural stabilization or suitable alternative methods utilizing materials which are more readily available in times of war than those used in the pitch-aniline-furfural process.

One of the more promising of these methods is the use of a stabilizer formed by combination of pitch with furfuryl-alcohol. Furfuryl-alcohol is relatively inexpensive and large quantities can readily be made available; it is compatible with pitch and hardens on treatment with an acid. Preliminary tests using small proportions of hydrochloric acid to promote hardening have indicated the possibility of developing satisfactory stabilizers by this method and further research is in progress.

Although construction difficulties have not yet been completely solved,

ne use of aniline-furfural and pitch-aniline-furfural combinations for stabilizing beach sands represents the most generally satisfactory method available at the present time. Its suitability for the stabilization of muds has not yet been determined.

3. The Thermite Method of Beach Sand Stabilization

In this method of beach sand stabilization, thermite and fluxing agents are used to fuse the sand grains together and form a hard coherent mass. The zones to be stabilized are prepared in four layers; the first (bottom) and third layers consist of thermite and the second and fourth layers of a mixture of sand and flux materials. The thermite is then ignited to fuse the grains together.

In the most recent tests conducted at Port Hueneme and described in a report by J. E. Halton and E. R. Holden (VE-1) the quantity of thermite in each of the two layers was 3 lbs. per sq. ft. A typical composition for one of the sand and flux layers in one of the more successful tests was:

| | |
|---|---|
| Sand | 26 lb. per sq. ft. |
| Borax | 1.5 lb. per sq. ft. |
| Hydrated Lime | 2.0 lb. per sq. ft. |
| Soda Ash | 3.0 lb. per sq. ft. |

Prior to ignition, the prepared zones were spread with water at the rate of 1/2 gallon per sq. ft. and allowed to stand for 1 hour. A test section, prepared and ready for igniton, is shown in Figure VE-12. The thermite was then ignited (see Figure VE-13), and immediately after completion of the thermite reaction, each zone was again sprayed with water until the surface was flooded (Figure VE-14).

Satisfactory results have not been obtained by the thermite method when the ground is not level because the thermite will not burn uphill. Furthermore, small inclines cause the molten phase to puddle and self-extinguishment results apparently due to insufficient heat transfer.

Difficulties have also been encountered due to discontinuous and incomplete combustion resulting from excessive moisture, too rapid combustion in the lower thermite layer and eruptions caused by the entrapment of gases formed during the high temperature reaction. The eruptions carried away the upper layers in many places, as will be seen in Figure VE-15, and in some cases sizeable craters were left along with a number of parallel cracks, two inches deep in places.

The eruptions were finally prevented by perforating the prepared zone, just prior to ignition, at 3-inch intervals with a 3/16-inch-diameter rod to a depth just above the bottom layer of thermite to vent out the gases formed during the reaction and by igniting the top layer of thermite 5 seconds in advance of the bottom layer. By this means, uniform and complete combustion of the thermite was obtained.

The texture of the stabilized soil, after aging for 16 hours, was very similar to that of a two-day old Portland cement mortar, being gray in color and fairly hard. With further aging it became light in color, very hard and granular. It was able to support a load approximately equivalent to a truck having a gross weight of 8/12 tons.

The cost of materials and labor in these more successful tests was about $3.29 per sq. ft. The cost of the materials alone was $3.09 per sq. ft., which is approximately six times that of 1/4-inch steel plate, while the weight of materials required was about twice that of 1/4-inch steel plate. This cost is also extremely high when compared with the 34 cents per sq. ft. cost of aniline-furfural stabilization.

As a result of these tests, it was concluded that by using a suitable construction procedure, a uniform and complete reaction, without discontinuities due to eruptions, can be induced in beach sands by the thermite method resulting in a stabilized material which can support traffic; however, erratic results will be obtained in inclement weather or on inclined surfaces. This latter fact makes the method unsuitable for stabilizing the sloping face of a beach.

In view of the undependable nature of the method, of the large amount of additives required and of the more promising results obtained in other methods of stabilization, it was recommended that investigation of the thermite method of beach stabilization should be discontinued.

4. The Calcium Acrylate Method of Soil Stabilization.

Soil solidification using calcium acrylate is a chemical stabilization process based on the formation of a complex system of soil particles interwoven in and attached to strong flexible ploymer chains by means of ion exchange followed by polymerization of the clacium acrylate monomer. A detailed description of the reaction and the properties of the treated soil have been presented in reports by the Soil Solidification Project at M.I.T. and the following is a modified form of a report by T. W. Lambe (VD-4) supervisor of the project.

Calcium acrylate ionises in the presence of water (either added with the acrylate or already in the soil) to form among other ions, a positively charged calcium acrylate ion. When the calcium acrylate is mixed with a base exchangeable soil, the calcium acrylate ions can become attached to the soil particles by replacing the anions of the soil particles in an exchange reaction; the displaced anions of the soil can react with negatively charged acrylate ions. The next step in the solidification process is to polymerize the acrylate, both that attached to the soil and that which remains dissolved in the water, to form long-chain polymers. This is accomplished by the addition of a catalyst (an oxidizing agent) and an activator (a reducing agent). The main polymer chains are attached to the soil through the calcium ions and are also cross-linked through the excess calcium ions. Thus a complex soil system, having considerable strength, is produced.

The stabilization of soils by ion exchange alone, which is the first step in the process described above, has been attempted previously. However, such reactions seldom increase the shear strength of a soil, although the replacement of a hydrophylic ion like sodium by a hydrophobic ion like calcium acrylate will reduce the water affinity of the soil. In stabilization of a soil by treatment with calcium acrylate, the contribution of ion exchange to strength is minor compared to that of the polymerization. In fact, calcium acrylate has been used successfully to stabilize soils which have no ion exchange capacities. For even these soils, there is evidence of attraction between the soil and the polymer. It has been suggested that this attraction could be caused by the

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electrically unsaturated ions on the surface of the soil particle drawing the ionic polymer, aided by some sort of secondary valency force between the soil and the polymer.

There are many possible variations in the actual procedure of stabilizing soils with calcium acrylate. The acrylate and the catalyst can be added separately or together; each chemical can be added in powder form, in water solution or in water slurry. The most desirable procedure would depend on the characteristics of the soil to be stabilized. A single powder consisting of an intimate mixture of acrylate and catalyst might be added to a wet soil to form a load bearing surface, for example, while a water solution of chemicals might be injected into a subsurface soil to increase its strength or decrease its permeability.

The amounts of chemicals and water required to stabilize a soil with calcium acrylate depend on the nature of the soil and the type of product desired. An increase in the percentage of calcium acrylate stabilizer increases the strength and flexibility of a soil and percentages varying from about 4% to 25% have been successfully used. However, if the amount of calcium acrylate exceeds about 15%, the cost would probably be prohibitive for most practical problems; below 4% the amount of chemical is so small that thorough mixing is difficult.

The redox catalyst system of ammonium persulfate as a catalyst and sodium thiosulfate as activator will satisfactorily effect the polymerization. A 1:1 ratio of persulfate to thiosulfate has been used with success. For acrylate treatments up to 10%, the amount of total catalysts does not appear to affect the quality or quantity of the polymer as long as there is enough to cause the polymer to form. The amount of catalyst, however, does have a marked effect on the gel time, which is the elapsed time between the addition of the catalyst and the start of polymerization. Percentages of catalyst and of activator from 1/2% to 20% of the acrylate weight have been used successfully and gel times in the neighborhood of 15 minutes have been obtained in field test sections.

The quantity of mixing water also affects the calcium acrylate reaction and the properties of the treated soil. For any soil there is a certain minimum amount of mixing water required for the polymerization reaction to occur. However, most natural soils which are not air dry appear to have enough water in them to meet this minimum requirement. The ease of mixing the soil, the compaction characteristics and the trafficability of the soil, on which the processing equipment must operate, depend on the water content, in addition to the strength, flexibility, shrinkage and freeze-thaw characteristics of the stabilized material. A decrease in the amount of mixing water causes a decrease in shrinkage and preliminary tests indicate that it reduces the susceptibility of the treated soil to freezing and thawing effects; however, the flexibility is also reduced by a reduction in mixing water. Maximum strength of the treated soil is attained at an optimum mixing water content.

After treatment with calcium acrylate, the shrinkage characteristics of the soil are reduced and the strength and flexibility are considerably increased. A sandy clay at the liquid limit can be transformed into a solid material with a tensile strength of about 80 lb.s per sq. inch, or with a water content of 16%, polymerization of a 10% acrylate treatment has increased the strength from 40 lbs. per sq. in. to 600 lbs. per sq. in.



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Calcium acrylate has been successfully used to solidify a wide variety of soils ranging from beach sands to highly plastic clays. The characteristics of the stabilized material depend on the composition of the soil. From the tests conducted to date, two general conclusions have been drawn:

(i) A stronger product is obtained if the soil contains some fine grained particles.

(ii) The soils with the higher liquid limits require more mixing water.

Soils which contain no small particles sometimes present a problem because they may not be able to hold the minimum amount of water desirable for the chemical reaction. Soils which contain a large proportion of particles of the expanding lattice type can present a mixing problem since much of the water added is taken up by the soil and is not available for the chemical reaction.

Investigations are continuously in progress to improve the calcium acrylate soil solidification process. The process can be improved by either the discovery of a better monomer or the addition of other monomers for copolymerization with the calcium acrylate. While the discovery of a monomer, which is superior to calcium acrylate is possible, it does not appear likely, since there are apparently certain requirements which limit the choice. Some of these requirements are that the monomer:

(i) Be an ionizable, water soluble salt of a polyvalent cation.

(ii) Form a polymer which is insoluble in water.

(iii) Be polymerizable under normal outdoor temperature and pressure.

(iv) Form a cross linking polymer.

(v) Exist in solid form.

(vi) Be composed of available (or at least, potentially available) components.

(vii) Be a substance which is, or could be, made at a reasonable price.

Many monomers have been and are being studied, but the calcium acrylate is the only one to date which fulfills these seven requirements.

The search for monomers which will improve the properties of calcium polyacrylate by copolymerization is under way. Two monomers have already been found which increase the strength of treated soil more than the calcium acrylate alone; another monomer has been found which increases the flexibility of treated soil.

Another way to improve the calcium acrylate treatment is to find a catalyst system which is better than the ammonium persulfate--sodium thiosulfate one. Encouraging progress is being made in this search.

Before calcium acrylate can be considered a practical method for the stabilization of beach soils, there are several problems which must be

overcome. Means, including techniques and equipment, of mixing the chemicals into the soil must be developed. Work is being done on this problem; although progress is being made, the solution is not yet at hand. More information on the behavior of soil treated with calcium acrylate is needed. Finally, calcium acrylate is, at present, expensive.

Since no company at present produces calcium acrylate on more than a pilot plant scale, no unit price can be given which has significance. The chemical cost is, of course, inversely related to the quantity of production. All that can now be said is that the use of calcium acrylate at pilot plant prices is uneconomical for any but special or experimental uses.

While soil stabilization by means of calcium acrylate requires further development, this method seems to offer the best possibilities of a practical procedure for the stabilization of muds.

5. References

V E-1: Halton, J. E. and Holden, E. R. - "Beach Sand Stabilization - Thermite Method" - Technical Memorandum M-011, U. S. Naval Civil Engineering Research and Evaluation Laboratory, Port Hueneme, California, 25 August 1950.

V E-2: Henley, R. H., Sieland, H. J. and Holden, E. R., - "Beach Soil Stabilization - Aniline Furfural Method"- Technical Report R-001, U. S. Naval Civil Engineering Research and Evaluation Laboratory, Port Hueneme, California, 21 September 1950.

V E-3: Mueller, W. A. - "Analine - Furfural Beach - Soil Stabilization" - U. S. Navy Civil Engineer Corps Bulletin, Vol. 5, No. 54, May 1951.

V E-4: Sandford, G. E. - "Beach Stabilization Tests, Report No. 1" - Advanced Base Proving Ground, unnumbered report, 14 April 1948.

V E-5: Winterkorn, Hans F. - "Second Monthly Progress Report on Phase II of the Beach Stabilization Research Program, NOy - 19300", Princeton, New Jersey, 1 October 1949.

V E-6: Winterkorn, Hans F. - "Final Report on Investigations of Beach Soil Stabilization Methods at Princeton, N. J. and Port Hueneme, California. Part II, July 1950.

V E-7: Winterkorn, Hans F. - "Final Report on Investigations of Beach Soil Stabilization Methods at Princeton, N. J. and Port Hueneme, California. Part III, July 1950.

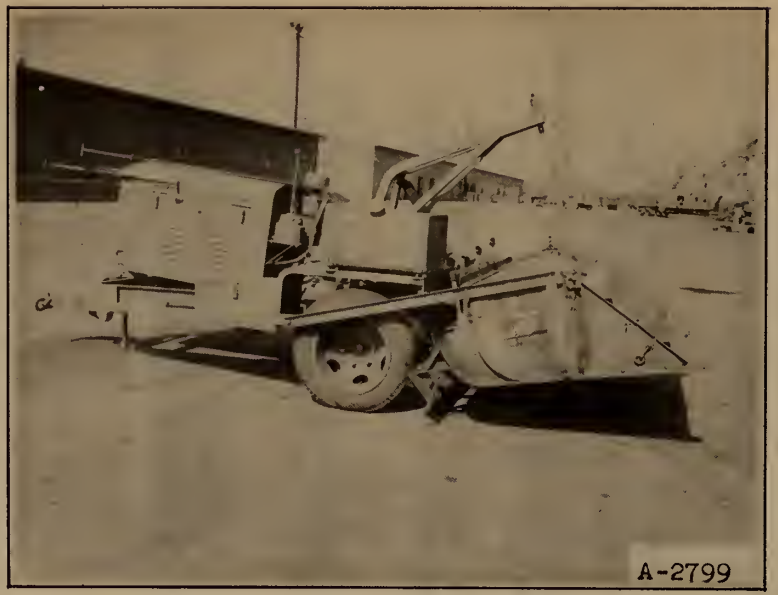



Fig. VE-1 - PULVI-MIXER (Official photograph, U.S.Navy)




Fig. VE-2 - Position of the spray bars. (Official photo, U.S. Navy)




Fig. VE-3 - Stabilizing train using the Seaman PULVI-MIXER. (Official photograph, U.S. Navy)




Fig. VE-4 - Cross-section of plot C-16 constructed with PULVI-MIXER, showing sand streaks. (Official photograph, U.S. Navy)



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Fig. VE-5 - P&H Single Pass Stabilizer (Official photograph, U.S. Navy)



Fig. VE-6 - Wood Roadmaster scoops up beach sand and passes it back through the pug mill. (Official photo, U.S. Navy)



Fig. VE-7 - Mounted on Athey tracks the complete unit is towed by a D-8 tractor. (Official photo, U.S. Navy)



Fig. VE-8 - View of spreader and densifier of the Wood Roadmaster.



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Fig. VE-11 - A strip of stabilized soil constructed by the Wood Roadmixer. (Official photo, U.S. Navy)



Fig. VE-9 - Finished strip supported impressive load of 38,600 pounds. (Official photograph, U.S. Navy)

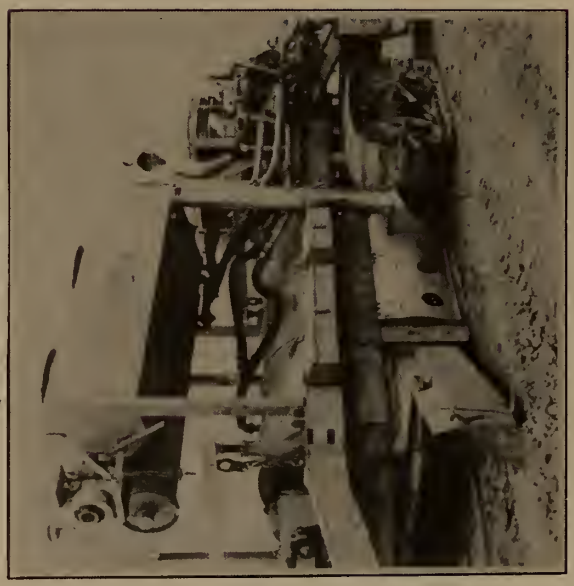


Fig. VE-10 - Improved model mounts 1,000-gallon elevated tank on the mixer chassis. (Official photo, U.S. Navy)




Fig. VE-12 - Plot I completed and ready for ignition. (Official photograph, U.S. Navy)




Fig. VE-13 - Ignition of Plot 3. (Official photograph, U.S. Navy)




Fig. VE-14 - Plot I being watered down while still wet. (Official photo, U.S. Navy)




Fig. VE-15 - Plot I during combustion. Note craters in foreground and eruptions taking place as the reaction proceeds towards the far end of the plot. (Official photo, U.S. Navy)

# F. MISCELLANEOUS METHODS OF CONSTRUCTING TEMPORARY ROADWAYS ON UNSTABLE BEACH SOILS

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## F. MISCELLANEOUS METHODS OF CONSTRUCTING TEMPORARY ROADWAYS ON UNSTABLE BEACH SOILS

At the present stage of development of beach soil stabilization, practical methods have not yet been developed for the satisfactory stabilization of all types and conditions of soil which may be encountered during an amphibious operation. In order that vehicles and equipment may be put ashore across non-trafficable beaches when soil stabilization procedures are not available, methods have been and are being developed for the construction of temporary bridges or roadways stretching from the landing craft to the stable areas on or beyond the backshore. In the normally accepted meaning of the term, soil stabilization is concerned only with the improvement of soil properties so that it becomes able to support applied loads. In this sense, the construction of a temporary roadway cannot be considered a method of soil stabilization. However, since the function of a temporary roadway is essentially the same as that of a stabilized soil, it is felt that a brief description of several types of temporary roadways should be included in this report.

Temporary roadways may be divided into two main types--pontoon causeways and lightweight mat roadways. Brief descriptions of these types of roadways are presented in the following pages.

### 1. Pontoon Causeway

The construction of a pontoon causeway is illustrated in Figures VF-1 to 4. The causeway was constructed in San Francisco Bay for the passage of heavy equipment from landing craft and ships, over mud flats, to a point near the grass line. Three pontoon assemblies were used, one extending seaward from near the grass line with the other two placed on each side and telescoped out to a point where the incoming landing craft were grounded.

An over-all view of the causeway from the seaward end is shown in Figure VF-1 and a view from the landward end in Figure VF-2. The pontoons are shown in Figure VF-3. The seaward ends of the seaward assemblies were equipped with cut-down sections, Figure VF-4, to facilitate the reception of landing craft ramps.

The pontoons floated on the incoming tide and at low tide; settlement in the mud due to the passage of heavy equipment was negligible.

### 2. Lightweight Roadways

Various types of lightweight roadways have been developed and are being investigated. A successful type of roadway developed during the last war and some recent developments are described below.

#### a. Acorn Laminated Mat

The Acorn laminated mat was designed to permit the passage of heavy equipment from beached landing craft, over mud or swamp to firm ground. The photographs illustrate the construction of aroadway during a test in San Francisco Bay in 1945.

A complete section of the matting measured 10' x 20' and had a thickness of 3". It consisted of 3 layers of 1" x 12" x 20' sheets, the upper and lower layers being laid at right angles to the center layers and the whole

being nailed together, (see Figure VF-5). Two layers of this matting were employed, the upper being staggered over the lower. 6' x 20', 2" welded aircraft landing mat mesh #4 gauges wire was laid over the top of the completed mat and was also placed between the two layers of mat at all points where the mat was waterborne at high tide. This prevented the lower section from being forced loose from the upper section by the passage of heavy equipment when the mat was waterborne.

The weight and cube per hundred feet of matting laid as above were 36 tons and 1000 cubic feet respectively. The weight and cube per hundred feet of wire mesh was 2-1/2 tons and 100 cubic feet. One hundred feet of completed matting was laid per hour using the following equipment:

1 T9 Austin Western Crane
1 Northwest Crane - M25
3 Freuhauf Trailers
1 Rubber-tracked T-9 Tractor

The personnel consisted of 35 men who were approximately equally divided between the mat-laying crew and the crew used for loading mats and mesh onto trailers from the LCT's.

The first section of the mat being placed is shown in Figure VF-6 and views of the completed roadway are shown in Figure VF-7 and 8.

It will be seen that this roadway is relatively heavy. Considerable progress has been made in recent years in developing lighter roadways, and these developments have been described in a recent issue of the Bureau of Yards and Docks Technical Digest. The following is reproduced from that report:

b. Swiss Roll (VF-1)

"Preliminary operational requirements laid down by the Bureau specify that the roadways must be capable of supporting moving loads of 60-ton tracked vehicles at speeds dictated by the character of the materials crossed. Roadway surfaces must provide at least 12 ft. of operational width and at the same time be no more than 19 ft. in total width so that the roadway may be adaptable for cargo ship side-carry.

"Operational requirements demand rigid specifications. For example, the roadway must be built to function over all kinds of treacherous surfaces and difficult subgrade, including solid rock, volcanic sand, mud and tidal water, typical conditions to be met in beach landings where ship-to-shore unloading proceedings are involved.

"One means of providing a safe and uninterrupted passage for combat vehicles over all kinds of terrain, now being developed at the Port Hueneme Proving Grounds, makes use of various modified versions of the British-developed prefabricated roadway, known as the 'Swiss Roll'. The Swiss Roll (shown in Figure VF-9) is essentially a lightweight, planked-over canvas mat designed to carry troops and light vehicles over water and mud. The light planks of slats are laid crosswise and secured, about 2 in. apart, to the canvas strip. The planks provide the traction surface. With the passage of moving loads, the surfaces of the roadway outside the ends of the planks swing up to become sides. This is accomplished by hinging the ribs which support these sides to the ends

of the treadway planks or slats. Additional longitudinal support for the roadway is gained by using several wire cables under the canvas at each wheel-track. When used as a ship-to-shore roadway, the cables, restrained at each end, are comparable to suspension cables.

"Thus far, in spite of modifications, the Swiss Roll is not entirely satisfactory over mud subgrades for even light vehicles. Trials with tracked and wheeled vehicles, weighing up to 15 tons, which moved as quickly as might be expected over such supporting structures, in almost every case resulted in failures at the joints of the roadway. Much mud was admitted to the inner surfaces at the failing points.

"In order to study the joint failures and to devise a means of preventing them, the Bureau's engineers have chosen to construct the roadway in 40-ft. lengths, instead of using the conventional 75-ft. sections. This will provide twice as many joints to observe in any given stretch of roadway during future tests.

"New and improved designs of this roadway now undergoing tests at Port Hueneme include the use of longitudinal plank tracks laid over the transverse planks. These will distribute the wheel-loads over a greater length of roadway and reduce concentrated sag. Another design, in which the use of prestressed lightweight concrete planking is embodied, is also being tested.

c. Pneumatic Roadway (VF-1)

"The proposed Pneumatic Emergency Roadway is highly portable and weighs 275 lbs. per linear foot; approximately one-fourth the weight of a similar length of the Navy Lightered Pontoon Causeway. It is estimated that the Pneumatic Emergency Roadway can be laid at the rate of one-quarter mile per hour and that it will carry moving vehicles weighing up to 60 tons.

"Basically, this unit consists of transverse metal channels, in which are collapsible rubber bags. The channels are pin-connected. As they are rolled out the bags are inflated to form pontoons 2 feet in diameter beneath the channels. Comments from field commands indicate that the proposed roadway lacks sufficient anchorage to secure it in some tides. Some field personnel, who have criticized the pneumatic roadway, point out that rubber pontoons are vulnerable to enemy fire and to rock- and coral-chafing and that their inherent stability, both transverse and longitudinal, is doubtful.

d. Styrofoam - Foam Plastic (VF-1)

"Styrofoam, an improved plastic foam, will justify continuation of experiments on the Pneumatic Emergency Roadway. It is a plastic substance which expands to many times its original size on contact with water. It has been proposed that small quantities of Styrofoam be installed in the collapsible bags that support the transverse metal channels of the Pneumatic Emergency Roadway. If styrofoam can be kept pliable enough to support the metal channels, it probably will permit uninterrupted vehicular traffic. In its present state, however, Styrofoam does not remain pliable. It dries up in a short time, becoming extremely brittle. In this form, it crumbles under weight and has no tensile or compressive strength. Therefore, the objective is to find a way to keep Styrofoam in a plastic or foamy state. This will give it the necessary compressive strength to support the required loads.

e. Dory-Shaped Sectional Roadway (VF-1)

"Preliminary plans for a proposed Dory-shaped Sectional Roadway indicate the prefabricated steel sections, about 40 feet long, will be assembled into 80-foot lengths at a staging area.

"An excellent feature of this type roadway is that the Dory-shaped sections will be nestable and can be launched one at a time. For example, an LST can side-carry twice as many Dory-shaped sections as pontoon causeways. This characteristic is preferable for tidal operations. Conversely, it presents quite a problem in bulk and weight for inland waterways and mud flats, where the sections must be transported by vehicles.

"The cross-section in the Dory-shaped roadway is about 20 feet wide with a 3-foot hollow area below the roadway deck. There will be 3-foot flared sides for added buoyance and wave protection. If added stability is needed, it is proposed to strap inflatable bags to the outside of the sections.

"In evaluating the worth of this plan, numerous disadvantages are cited. For instance, the end connections of the 80-foot sections present difficult design problems. Also, the anchoring problems are similar to those of the rubber pontoon system. In addition, the fittings for side-carriage and launching may prove too complicated to be practical. And the unit weight also presents a problem of mobility."

3. References

V F-1: Bureau of Yards and Docks - "Bu Docks Engineers are Testing Light-weight Roadways for Use in Amphibious Landings" - Bureau of Yards and Docks, U. S. Navy, Technical Digest. No. 17, May 1951, pp. 3-5.



Fig. ∇F-1 - Overall view of causeway. LSM approaching left causeway. (Official photographs, U.S. Navy)

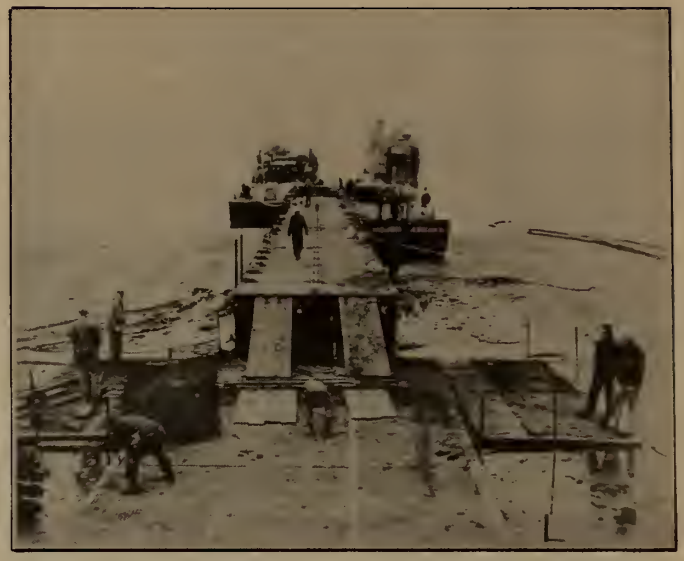


Fig. ∇F-2 - This photo shows that a sloped cut-down pontoon section would be desirable at the shoreward end of the causeway. (Official photo, U.S. Navy)



Fig. ∇F-3 - Medium tank made repeated trips over causeway to determine whether pontoons would be stuck in mud. They floated free on the incoming tide. Note negligible settlement after 40 tank passages. (Official photo, U.S. Navy)

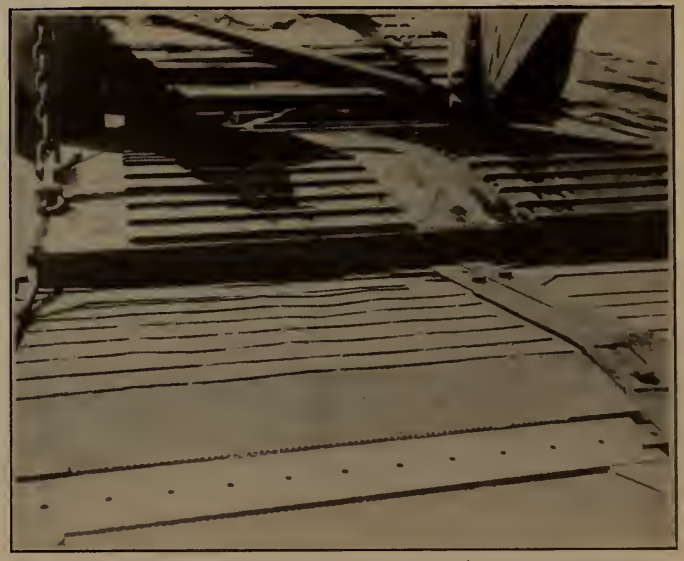


Fig. ∇F-4 - Landing craft ramp on cut-down section (Official photo, U.S. Navy)



Fig. ∇F-5 - If mat is being laid in water or over obstacles, a 4" x 8" medium gauge sheet of iron, with 4 holes 1" in from each corner, is used. (Official photograph, U.S. Navy)



Fig. ∇F-7 - 500' of matting was laid at the rate of 100' per hour. 160' was laid over mud flat, remainder through heavy tule growth. (Official photogroph, U.S. Navy)



Fig. ∇F-6 - First mat being dropped in place for mat heading. Wire mesh is laid on mud to provide initial footing for men. (Official photo, U.S. Navy)



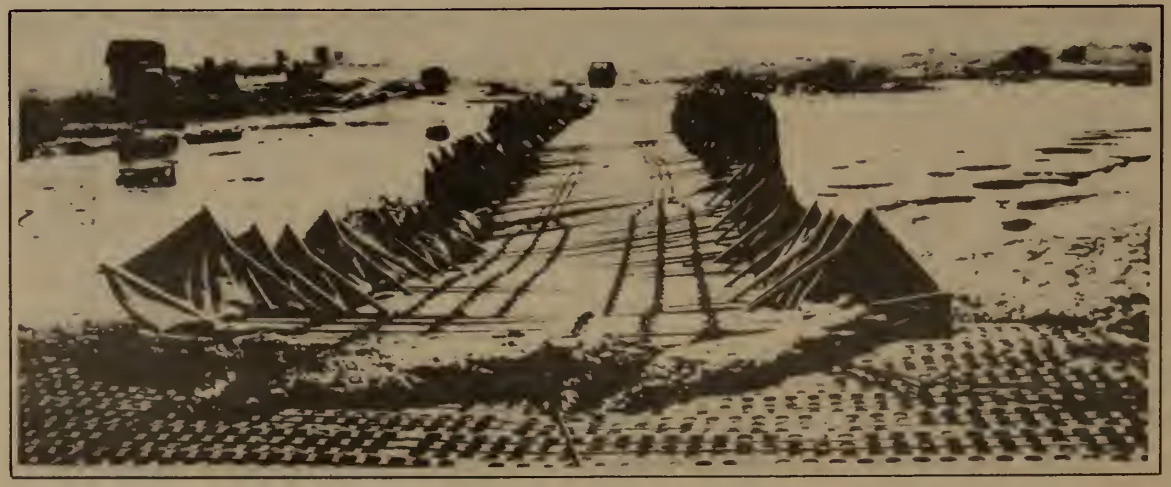
Fig. ∇F-8 - Mat laid from shoreward end of pontoon causeway; note spuds used to anchor Acorn mat when floated by incoming tide. (Official photo, U.S. Navy)



a. Loading a section of "Swiss Roll" lightweight roadway at the proving ground.



b. The "Swiss Roll" gets a dry run.



c. The "Swiss Roll" lightweight roadway laid across a tidal flat, ready for a trial run of heavy equipment.

FIGURE VF-9 - THE SWISS ROLL

(Official photographs, U.S. Navy)



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G. MISCELLANEOUS

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## G. MISCELLANEOUS

### 1. Acknowledgment

Many sources of information have been used in the preparation of this manual on soil trafficability and stabilization. While it is not possible to acknowledge all of these sources individually, except by the inclusion of references wherever possible in the text, it is felt that a special acknowledgment should be made for the use of information on the trafficability of soils contained in reports by the Waterways Experiment Station, Vicksburg, Mississippi, and of a report on beach sand stabilization prepared by the Soil Consolidation Task Committee of the National Security Industrial Association. These reports have aided materially in the preparation of the manual. Special acknowledgment should also be given to members of the staff of the U. S. Naval Civil Engineering and Evaluation Laboratory for the material in their reports and for kindly furnishing many original photographs for use in this manual.

### 2. Glossary of Terms Used in Part V

| | |
|---|---|
| Apparent Cohesion: | That part of the shear strength of a soil which is due to water menisci. |
| California bearing ratio: | An index of the bearing capacity of a soil which is used in the design of flexible pavements; determined by a standard penetration test. |
| Coefficient of Uniformity: | The coefficient of uniformity is the ratio of the 60% size to the 10% size of a soil sample (see Effective Size). |
| Compaction: | The process of reducing the void ratio of a soil by the expulsion of air from the voids. |
| Cone Index: | The force in lbs. at which movement just starts when the U. S. Corps of Engineers cone penetrometer is pushed into the soil. |
| Cone Penetrometer: | A portable penetration device consisting essentially of a standard size cone at the end of a rod. The resistance of a soil to penetration of the cone is used as an index of the bearing capacity and trafficability of the soil. |
| Consolidation: | The process of reducing the void ratio of a soil by the expulsion of water from the voids. |
| Densification: | The process of increasing the dry density of a soil. |
| Density of Traffic: | The number of passes of vehicles over an area. |
| Drawbar Pull: | The pull which a vehicle can develop in addition to moving itself. |
| Dry Density: | The weight of solids per unit volume of soil. |

**Effective Size or Effective Diameter:** A grain size distribution curve for a soil sample is usually plotted to show grain diameter as abscissae and percentages of grains (by weight) finer than these diameters as ordinates. The effective size is that diameter corresponding to an ordinate of 10% on the grain size distribution curve. This diameter $D_{10}$ is also called the 10% size. The 60% size $D_{60}$ is the diameter corresponding to an ordinate of 60% on the grain size distribution curve.

**Liquefaction:** The collapse of the grain structure of the soil causing the soil to behave temporarily as a liquid.

**Liquid Limit:** Water content of soil when consistency is at an established boundary between semifluid and plastic states.

**Mud Packing:** The accumulation of soil on track rollers or beneath the fenders or carriage of a vehicle causing an increase in rolling resistance.

**Muds:** Fine-grained soils having water contents ranging from about the plastic limit to somewhat greater than the liquid limit.

**North Dakota Cone Bearing Value:** An index of the bearing capacity of a soil determined by a penetration test using the North Dakota cone apparatus.

**Plastic limit:** Water content of soil when consistency is at an established boundary between semisolid and plastic states.

**Plasticity Index:** Liquid limit minus plastic limit.

**Rolling Resistance:** The resistance to forward motion of a rolling wheel or vehicle. External rolling resistance is caused by the soil on which the vehicle is rolling. Internal rolling resistance is caused by mechanical features of the vehicle, mainly, friction in the axle bearings.

**Slippage:** Relative motion between a wheel or track and the supporting soil at the points of contact.

**Soil Stabilization:** The process of improving the properties of soils by physical or chemical means in order to increase their resistance to deformation under applied loads.

**Soil Truss:** An instrument for measuring the shear strength of a soil in situ.

| | |
|---|---|
| Static Loading: | Loads built up to their maximum value over periods greater than three minutes. |
| Trixotropy: | The phenomenon of stiffening of gels with time. |
| Transient Loading: | Loads built up to their maximum value over periods shorter than thirty seconds. |
| Void Ratio: | Ratio of volume of voids in soil to volume of solids. |
| Water Content: | Ratio of the weight of water to the weight of solids in a soil. |
| Water Plasticity Ratio or Percent in Plastic Range: | $\frac{\text{Water content of soil - plastic limit}}{\text{Liquid limit - plastic limit}}$ |

3. List of Symbols Used in Part V

| | |
|---|---|
| b | width of loaded area on soil. |
| c | unit cohesion or effective cohesion of soil. |
| c' | $n \gamma \tan \phi$ |
| d | depth of loaded area below adjacent soil surface. |
| $d_t$ | diameter of capillary tube. |
| e | base for Naperian logarithms. |
| f | coefficient of friction between track or tire and soil. |
| $f_1$ | coefficient of friction between tracks or tires of towing vehicle and soil. |
| $f_2$ | coefficient of friction between tracks or tires of towed vehicle and soil. |
| h | height of grouser. |
| $h_c$ | height of capillary rise. |
| k | slip factor; $k = 1$ (no slippage); $k = 0$ (100% slippage between wheel and soil). |
| m | impact factor. |
| n | factor used in Terzaghi modification of Prandtl bearing capacity formula. |
| p | normal pressure. |
| $p_r$ | mean horizontal resistance pressure of soil to movement of wheel, over full frontal area of ground resistance. |

$p_v$ mean intensity of vertical ground pressure on rolling wheel.

$p_z$ horizontal pressure (perpendicular to direction of motion) on an element of soil immediately below a rolling wheel.

$q$ unit pressure on loaded area of soil.

$q_u$ ultimate bearing capacity of soil for static loads.

$q_{ud}$ ultimate bearing capacity for static loads of sand on dry berm of beach.

$r_t$ radius of capillary tube.

$s$ shear strength of soil.

$s_g$ spacing of grousers on track.

$u$ tensile stress in water due to capillary effects.

$v$ peripheral velocity of wheel or track.

$w$ width of tire or track.

$w_{cc}$ lowest water content of soil at which cone index is independent of compactive effort.

$w_{co}$ water content of soil at which, for a given compactive effort, cone index is a maximum.

$w_d$ effective unit weight of soil on dry berm of sandy beach.

$w_{do}$ water content of soil at which, for a given compactive effort, dry density is a maximum.

$w_{dc}$ lowest water content of soil at which dry density is independent of compactive effort.

$z$ depth of sinkage of rolling wheel in soil.

$A$ adhesion between track, wheel or wheels of vehicle and soil.

$A_1$ adhesion between tracks or tires of towing vehicle and soil.

$A_2$ adhesion between tracks or tires of towed vehicle and soil.

$D$ diameter of wheel.

$F$ tractive force.

$F_h$ $0.64 \frac{h}{w} \cot^{-1} \frac{h}{w}$.

$F_s$ $0.64 \frac{s_g}{w} \cot^{-1} \frac{s_g}{w}$.

$F_t$ maximum value of tractive force when limited by traction.

H available drawbar horsepower of vehicle.

L Length of contact of track - inches.

N normal force between wheel and ground.

$N_c$ dimensionless bearing capacity factor depending only on value of $\phi$ for soil.

$N_q$ dimensionless bearing capacity factor depending only on value of $\phi$ for soil.

$N_\gamma$ dimensionless bearing capacity factor depending only on value of $\phi$ for soil.

$N'_c$ dimensionless bearing capacity factor depending only on value of $\phi$ for soil.

$N'_q$ dimensionless bearing capacity factor depending only on value of $\phi$ for soil.

$N'_\gamma$ dimensionless bearing capacity factor depending only on value of $\phi$ for soil.

$N_\phi$ $\tan^2 (45 + \frac{\phi}{2})$.

P drawbar pull.

$P_1$ maximum drawbar pull developed by vehicle on level ground.

R rolling resistance.

R' horizontal resistance of soil to movement of wheel.

$R_1$ rolling resistance for towing vehicle.

$R_2$ rolling resistance for towed vehicle.

$R_{2s}$ sliding resistance of towed vehicle other than friction and adhesion.

$R_{2s\ max.}$ value of $R_{2s}$, when weight of towed vehicle is $W_2$ max.

$T_s$ surface tension of water.

V load on wheel or track.

W weight of vehicle plus load, or wheel load for vehicle.

$W_1$ weight of towing vehicle plus load.

$W_2$ weight of towed vehicle plus load.

$W_{2\ max.}$ weight of towed vehicle at which towing vehicle starts to slip.

$\alpha$ slope of ground.

$\alpha_{max.}$ maximum slope which vehicle or vehicles can negotiate.

$\gamma'$ effective unit weight of soil.

$\gamma_w$ unit weight of water

$\phi$ angle of internal friction or effective angle of friction of soil.

$\phi_d$ angle of internal friction of sand on dry berm of beach.

$\theta$ variable angle.

# MANUAL

# OF

# AMPHIBIOUS OCEANOGRAPHY

## SECTION VI

## INTELLIGENCE FOR AMPHIBIOUS OPERATIONS

CLASSIFICATION CANCELLED OR CHANGED TO CONFIDENTIAL
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# MANUAL OF AMPHIBIOUS OCEANOGRAPHY

## SECTION VI: INTELLIGENCE FOR AMPHIBIOUS OPERATIONS

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# MANUAL OF AMPHIBIOUS OCEANOGRAPHY

## SECTION VI - INTELLIGENCE FOR AMPHIBIOUS OPERATIONS

### A. INTRODUCTION

BY M. P. O'BRIEN

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## A. INTRODUCTION

Ocean waves are generated by winds at sea, frequently at distances of thousands of miles from the shores on which they break. The velocity, fetch, duration, and direction of the wind determines the height, period, and direction of the waves generated. These characteristics of the incident waves and the bottom contours of the shoreline determine the character of the surf at any time, and are predictable with an accuracy comparable with that of other meteorological phenomena. The probably effect of surf on landing craft has been investigated and can now be predicted to a fair degree of accuracy. It is the purpose of this introduction to discuss qualitatively surf and related phenomena in order that officers not concerned with technical phases of the subject may know what information should be available to them in advance of, and during, a landing operation.

### 1. Origin and Nature of Ocean Waves

It is a matter of everyday knowledge that wind blowing over a water surface causes waves, light breezes causing ripples and small waves, and strong winds causing waves of considerable length and height. A fact which has masked the relationship between wind and resultant waves is the occurence of waves, frequently very large ones, when the wind velocity at the point of observation is either light or in a direction different than that of wave advance. This situation tended to obscure the quantitative relationship between the intensity of the generating wind and the resultant waves; the underlying reason being that waves travel great distances once generated and that the local wind merely sets a lower limit for wave size. It is obvious that, to forecast breakers at a selected beach, it is necessary to know the wind velocity, duration, and direction over all ocean areas from which waves might originate. Figure VI A-1 is a typical weather chart of the north Pacific Ocean on which are shown the lines of constant atmospheric pressure, isobars, from which wind velocities and directions can be estimated. Several facts about such charts and their use in wave forecasting are important in appraising the accuracy of forecasts made from them:

(i) The charts are usually based upon a relatively small number of pressure measurements and, over large areas, the position of the isobars is known only approximately.

(ii) The wind velocity at the ocean surface is smaller than that in the upper levels of the atmosphere and the ratio between these two velocities is not constant.

(iii) The winds do not follow a straight path; the borderline of the generating area is not sharply defined.

(iv) The surface of the ocean is already distrubed by waves from other areas.

(v) The wind velocity and direction change continually.

These circumstances make wave forecasting an art as well as a science, but aerologists experienced in the technique are able to achieve a high degree of reliability provided that reliable and complete weather charts are available.

Figure VI A-2 shows aerial photographs of two areas of a lake (Clear Lake, California) in which waves were growing under the action of winds. The distance from the windward shore to the first area is 2.3 nautical miles and with a wind velocity of 6.4 knots while the distance from the windward shore to the second

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area is 4.9 natuical miles with a wind velocity of 11.0 knots. These photographs illustrate several features of wave growth, as follows:

(i) Waves lengths and heights increase as the waves travel under the driving force of the wind.

(ii) Wave height, length, and direction are not uniform.

(iii) The length along the crest is finite, increasing as the distance between successive crests (wave length) increases.

Figure VI A-3 is a typical distribution of wave length, as measured from the area just shown at 4.9 miles fetch. Although there is variability in all characteristics, there is a statistical average which shows a regular variation with wind velocity, fetch, and duration.

The limited fetch measurements at Clear Lake previously described were not complicated by the presence of waves from other fetches. Figure VI A-4 shows a record of wave pressures taken at sea, using instruments recently developed for this purpose. It is apparent that height and period are variable. These waves were generated by the winds shown in the weather chart of Figure VI A-1.

Figure VI A-5 is an aerial photograph of waves near a generating area (Formosa). In general features, the waves are like those at Clear Lake but on a large scale. Wave length and direction of travel are variable, and the waves have a finite length along the crest.

Figure VI A-6 shows a set of swells which were generated somewhere in the "Roaring Forties" and the "Furious Fifties" on the southern hemisphere. Greater uniformity of wave length and height and relatively greater lengths along the crests stand out in comparison with Figure VI A-5 and are characteristic of waves from distant generating areas. (Figure VI A-6 is Santa Maria River, California $H_b$ = 14.5 ft. T = 16.2 sec).

The preceding discussion has been of a qualitative character and has, perhaps, tended to overemphasize the uncertainties and irregularities of wave generation and travel. It is pertinent at this point to state some of the quantitative relationships which have been established by theory and confirmed by observations in deep water:

Wave Velocity (ft./sec.) = 5.12 x (period in seconds)

Wave Length (ft.) = 5.12 x (period squared)

Recent studies show that the wave height generated by a continued wind blowing over a long fetch is

Height (ft.) = 0.026 x square of wind velocity in knots

These facts, plus the limitations imposed by duration and fetch, appear in Figure VI A-7 which represents the basic relationships used by wave forecasters.

Figure VI A-8 presents the curves used by forecasters for the decay area (i.e., the area between the end of the fetch and the shore for which forecasts are being made).



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2. Sources of Wave Information

Waves are an aspect of weather and like weather show average values over long periods which may be designated as the "wave climate" of a certain area of the ocean. Climate has to do with such questions as "what are the normal temperature and wind velocity on the west coast of Korea in September?" On the other hand, weather information deals with questions such as "what is the temperature likely to be tomorrow?"

Statistics on wave characteristics are not as numerous or reliable as other climatic data but some information is available through the "Wave and Swell Charts" of the U. S. Navy Hydrographic Office which are based on visual estimates of wave height, length, and direction entered in the logs of vessels. By averaging the observations in selected areas by months, sea and swell charts have been prepared. From these charts, officers planning an amphibious operation may obtain an indication of the average wave conditions off a selected beach and of the chance that extreme conditions may occur.

Another approach is through the "Historical Weather Maps" redrawn for each day for the last forty odd years. From these maps, the waves may be "hindcast" for selected areas for each day and then may be analyzed to obtain average and extreme conditions by months.

The most accurate information is that which is obtained from wave recorders. Unfortunately there are but a few in operation, and these have not operated continuously. Wave recorders have been installed at Guam, M.I; Pendeen, England; Bermuda; Pt. Sur, California; Heceta Head, Oregon; Camp Pendleton, California; Cuttyhunk, Massachusetts; and a few other locations.

This "climate" type of wave information in valuable in making plans but it is desirable to go beyond what has occurred in the past and to forecast the wave conditions likely to exist on D-day and subsequently during the cargo phase. The information necessary to make this forecast has been discussed previously in part but there are additional aspects, particularly the forecast interval, which should be mentioned.

Individual waves travel at a velocity given by the relationship $C = (5.12T^2)$ but the advancing front of a change in wave conditions moves only at half this velocity. As an approximate indication of the interval, consider waves of 10 second period traveling 1000 nautical miles to a particular shore. The velocity is 51 feet per second or 31 knots. The forecast interval would be approximately $\frac{1000 \times 2}{3.} = 61$ hours. There is a possible increment of this wave-travel interval by the interval that the aerologists can forecast the winds, using synoptic weather maps. If the fetch is close to the beach, the interval is short and vice versa. Longer range forecasts can be made by using prognostic weather maps, with the accompanying decrease in accuracy.

Deep water wave recorders have been developed to detect the long low swells, which are frequently not observed visually and which are the waves dominating the surf zone. In the previous example two hours warning of a preceptible increase in wave height might be obtained by placing a recorder 30 to 40 miles offshore.

Persistence is an aid to weather forecasting of all types. Considerable accuracy may be achieved over a 24-hour interval by merely forecasting that tomorrow will be like today. When a change occurs this amateur approach to

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forecasting is in error but in many localities it is 80 to 85 percent correct. The same situation exists in regard to waves. By recording waves at sea, a short-range highly accurate forecast may be made from the travel time of the wave fronts; from persistence along, this forecast interval may be extended. Wave measurements at sea and the forecasting technique combined will provide highly reliable forecasts.

3. Hydrography of Sandy Shorelines

For obvious reasons, shorelines composed of sand, gravels or mud are generally selected for landings. Where heavy surf occurs, it usually removes the mud from the shoreface and deposits it offshore, leaving as the beach proper, sand or gravel, or both, depending on the source of the material forming the beach. This circumstance is fortunate, in that soft, muddy shores are not likely to be encountered where surf may also cause difficulty. The following notes on beach hydrography deals with sand and gravel beaches.

Figure VI A-9 shows selected beach profiles drawn to a common scale. It is to be noted that the scale is distorted 20 times, the vertical dimensions being exaggerated in comparison with the horizontal.

Studies of beach characteristics have not progressed as far as they might have but there are some justifiable generalizations pertinent to amphibious operations:

(i) The general slope of a beach increases with the size of the sand composing it. Flat beaches are composed of fine sand and steep beaches of coarse sand or gravel (Figure VI A-10).

(ii) Beaches of fine sand, that is flat beaches, are generally hard-packed and firm below the limit of uprush at high tide. They are easy to traverse.

(iii) Beaches of coarse sand or gravel are generally soft and difficult to traverse.

(iv) The shoreface is highly mobile and responds quickly to changes in waves and currents. Beach profiles show seasonal changes, the visible beach generally being cut back under storm waves and filling during periods of low swells.

(v) Erosion of the shoreface under storm wave action is accompanied by bar formation offshore.

(vi) Beaches composed of coarse sand generally show greater variations in profile along the shore than do those composed of fine sand.

The problems of beach intelligence have been given considerable attention recently by engineers and geologists and a fairly complete representation of the character of an enemy-held beach may be developed from aerial photographs, charts, and other sources. (Aerial methods of surveying beaches will be discussed later.)

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4. Surf

As ocean waves move shoreward they experience a transformation involving the following components:

(i) The velocity is reduced

(ii) The length decreased

(iii) The height increased

(iv) The wave crests turn to reduce angularity with the bottom contours

(v) The steepness, or ratio of height to length, increases to a limiting value at which the wave "breaks"

(vi) The depth at breaking is approximately 1.3 times the height.

These features are represented in the aerial photograph of Figure VI A-11. Figure VI A-12 shows several successive stages of the movement of a breaking wave, both from the air and from the shore.

Much more could be said regarding the technical aspects of the motion of waves in shallow water. They move in accordance with well established laws and their behaviour is not predictable with a fair degree of accuracy. Figures VI A-13 to 16 show a range of surf conditions.

Waves transmit energy which is dissipated in the breakers and the surf zone. The energy of a wave is proportional to the square of the product of height and period ($H_o^2 \times T^2$). If two waves have the same height in deep water but one has twice the period of the other, the wave of longer period carries four times as much energy and its "break" is correspondingly more violent. It is for this reason, that long low swells, frequently imperceptible to aerial observers and to ships at sea, dominate the surf. Figure VI A-17 shows this phenomenon clearly; the long swells being obscured by shorter wind waves in deep water but "peaking up" as they approach the line of breakers to obliterate the other waves. Several sets of waves are commonly observed and it is imperative that measurements made for amphibious operations select those important in creating surf.

The velocity of a wave decreases in shallow water and this effect accounts for the bending of waves in such manner that their crests tend to become parallel to the shore. This phenomenon is known as refraction. It is illustrated in Figure VI A-18 which shows the breakwater at Half Moon Bay, California and the shoreline to the south. Long waves "feel the bottom" at greater depths than do short waves with the result that the long period swells swing around the point and reef to break on the shoreline near the pier. Short waves due to local wind would continue on past the point to the further shore. Figure VI A-19 shows the effect of wave refraction at Moss Landing, the deep canyon directly offshore from the pier having the effect of reducing the intensity of the surf. The refraction of waves may be predicted with considerable accuracy and such studies should be made in the advance planning of an operation for the wave directions and periods likely to cause difficulty in the operation of boats and vehicles.

The principle of the conservation of energy applies to waves. From it can be derived the effect of refraction on wave height because as wave crests become longer through refraction, the wave height must decrease. Figures VI A-20 to 22 show the aerial photographs, refraction diagrams, and measured and predicted

wave heights along the shore near Monterey Harbor. Measurements were made along the shore and at sea to check the method.

5. Along Shore Currents

The breaker projects forward a mass of water having considerable momentum. If the crest of the breaking wave is at an angle with the shoreline, this momentum has a component parallel to the shore which tends to generate a current. Figure VI A-11 shows waves breaking at an angle on the beach at Camp Pendleton, California. Figure VI A-23 shows the general relationship developed from laboratory experiments, current measurements on the surface, and aerial photographs of color bombs dropped from planes. Waves of the same height, angle, and period develop stronger currents on steep beaches than on flat ones. Below some not very well defined combination of beach slope and wave characteristics, the currents in the surf zone break seaward at rather regularly spaced distances along the shore (surf rips).

The alongshore currents may be predicted for approximately straight shorelines but nothing quantitative is known about them on curving or irregular bottoms such as that at Moss Landing. The aerial methods used to obtain current data are also applicable to reconnaissance of enemy-held beaches.

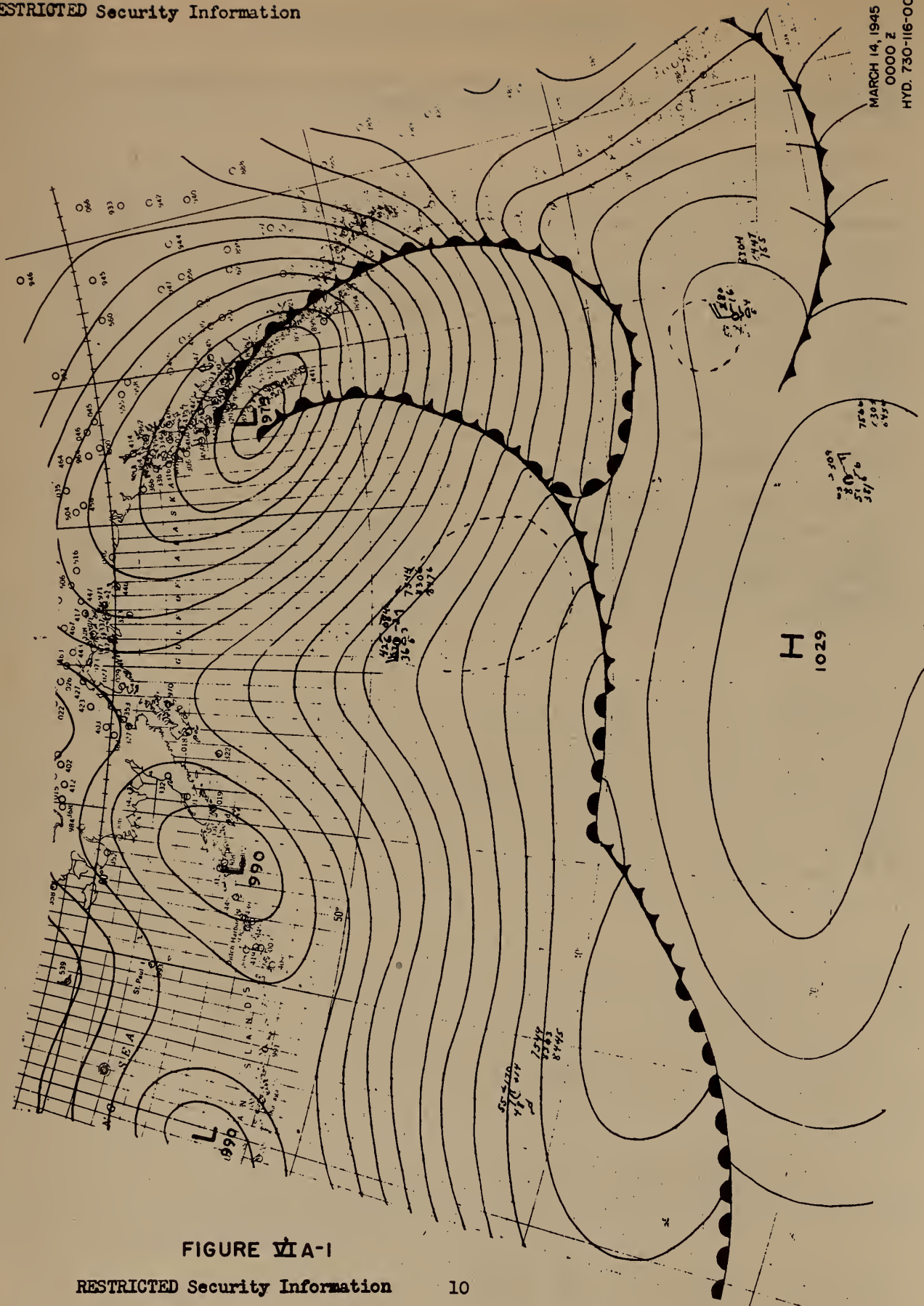

MARCH 14, 1945
0000 Z
HYD. 730-116-00
H
1029
L
976
L
990
L
998
SEA
ISLAND

FIGURE VI A-1



a. Limited Fetch Study, Clear Loke, 2/2/44. Wind velocity = 6.4 knots*, Fetch=2.3 nm, Wave length=8.2 ft, Wave height=ripples



b. Wind velocity=11.0 knots*, Fetch=4.9 nm, Wave length=23.7 ft, Wove height=0.6 ft (approx)

* Wave height varied from 0.96' to 0.60' from 1430 to 1800 for respective wind velocities of 16.4 knots to 11.0 knots at DUKW, while on shore the velocity varied from 10.5 to 5.6 knots, respectively.

FIGURE VI A-2

MEAN WAVE LENGTH = 18.8' ± 0.14'
$\sigma_m$ = 2.7'
MEAN WAVE LENGTH
LONGEST 30% = 23.7' ± 0.16'
$\sigma_m$ = 1.8'
FREQUENCY OF OCCURRENCE
0
10
20
30
40
50
60
70
80
WAVE LENGTH IN FEET
10
15
20
25
30
CLEAR LAKE, CALIFORNIA
UNIVERSITY OF CALIFORNIA
FLUID MECHANICS LABORATORY
BERKELEY
DATE 2-1-45 DRAWN BY MOREY
HYD. 506-116-00
PHOTO No. 8
407 OBSERVED WAVES



FIGURE VI A-3

LOADING #10 - 14 MAR 45

NELLIE #2

IMMERSE 0853

#52

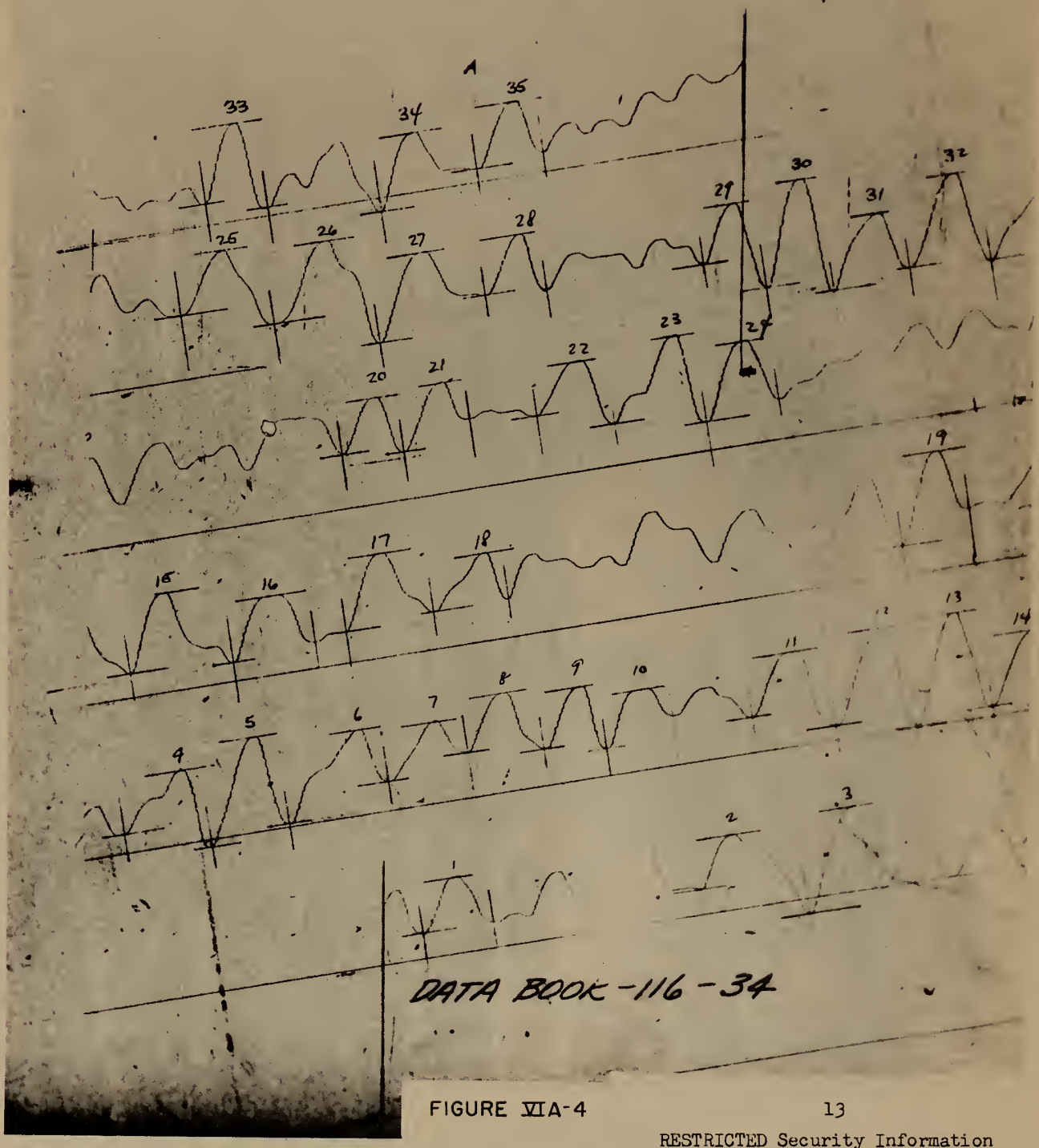



FIGURE VIA-4



Photo No. 5467. Refraction effects on Formosa. Note the effect of submarine gullies near shore on wave front.

FIGURE VIIA-5



FIGURE VII A-6 - Waves from a storm from the "Roaring Forties" or the "Furious Fifties" of the Southern Hemisphere. - Location south of Santa Maria River, California. $H_b$=14.5 feet, T=16.2 seconds.

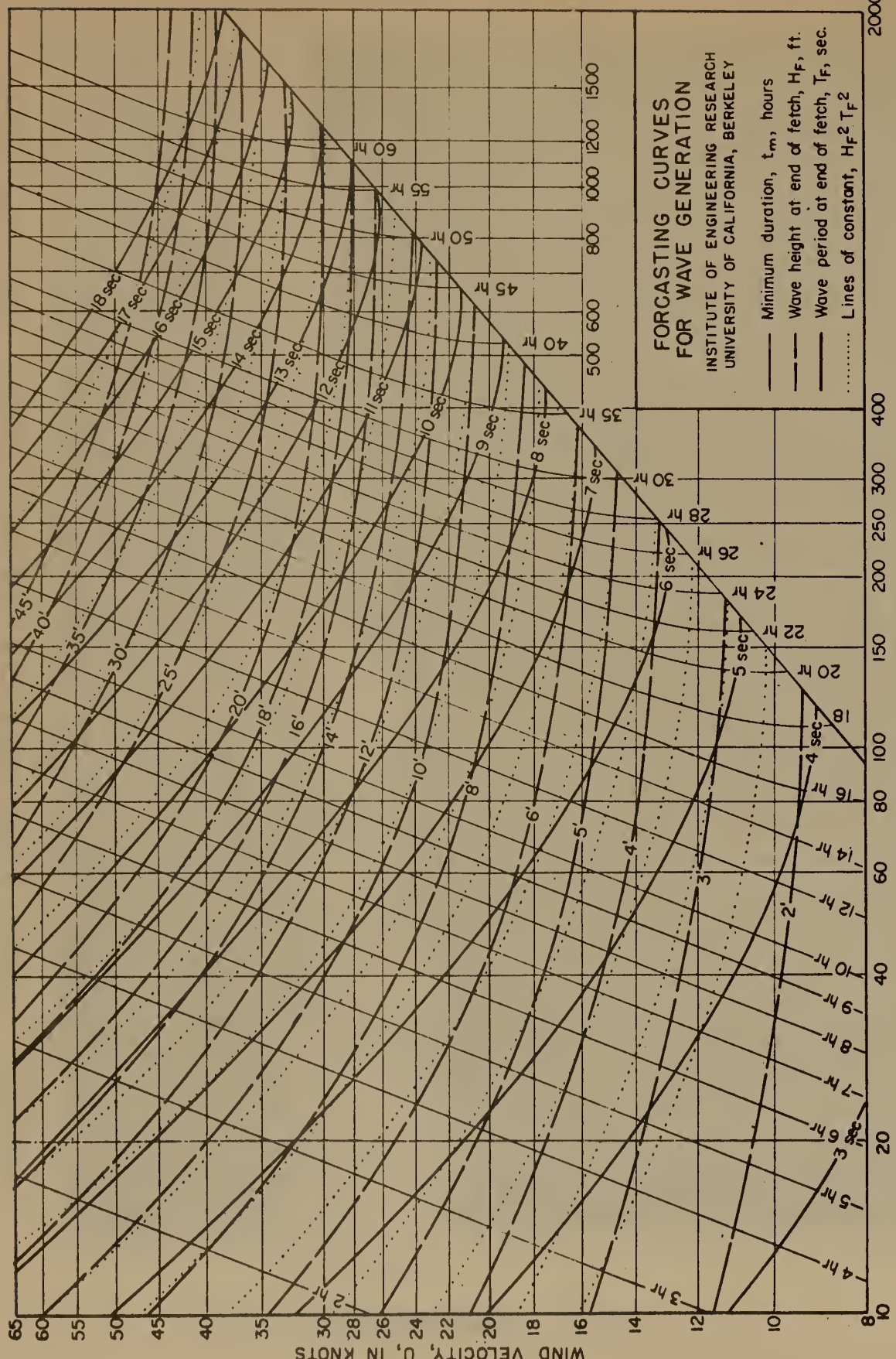

FORCASTING CURVES
FOR WAVE GENERATION
INSTITUTE OF ENGINEERING RESEARCH
UNIVERSITY OF CALIFORNIA, BERKELEY
Minimum duration, $t_m$, hours
Wave height at end of fetch, $H_F$, ft.
Wave period at end of fetch, $T_F$, sec.
Lines of constant, $H_F^2 T_F^2$
WIND VELOCITY, U, IN KNOTS
FETCH DISTANCE, F, IN NAUTICAL MILES

FIG. XI A-7

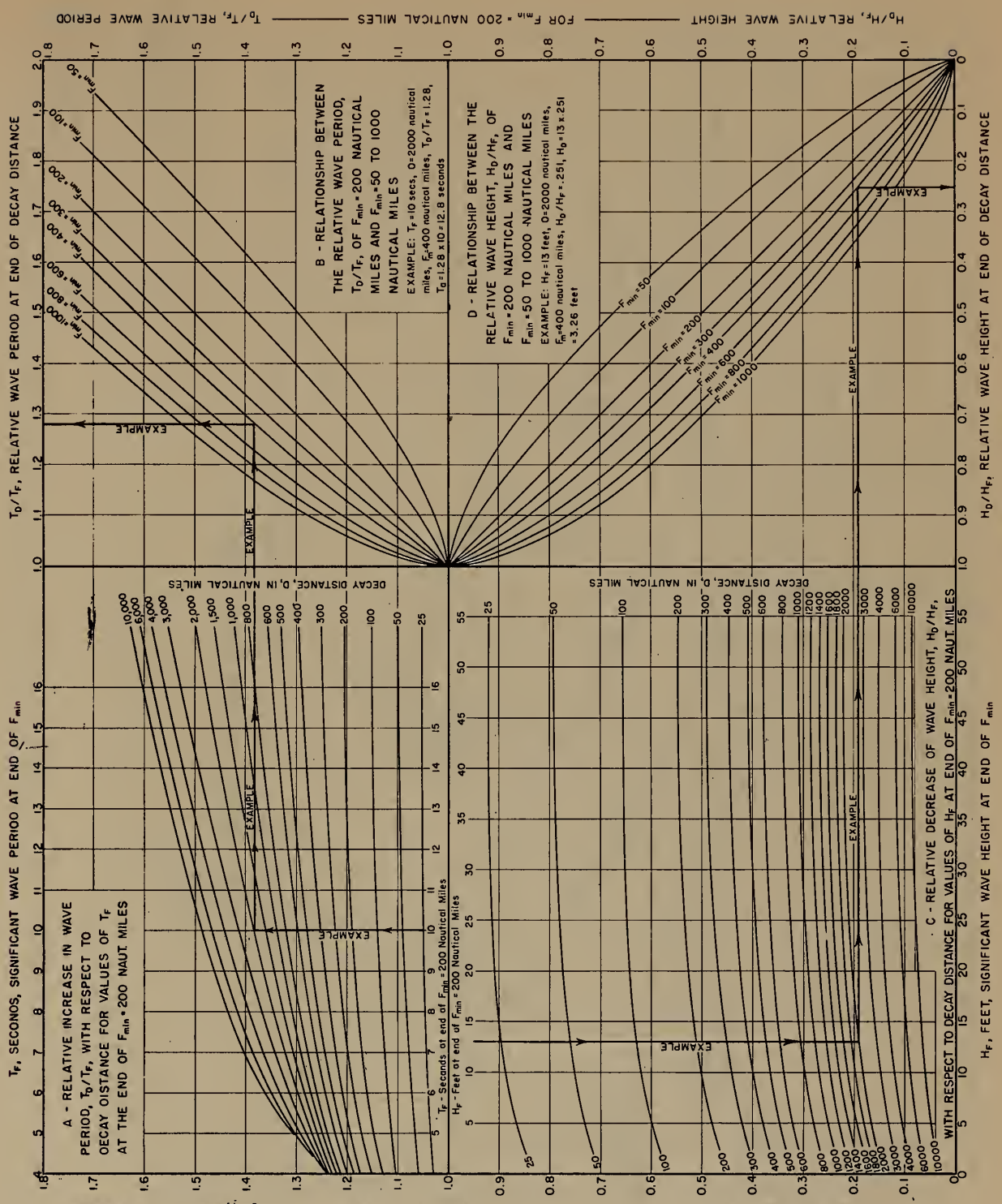

A - RELATIVE INCREASE IN WAVE PERIOD, $T_D/T_F$, WITH RESPECT TO DECAY DISTANCE FOR VALUES OF $T_F$ AT THE END OF $F_{min}$ = 200 NAUT. MILES
B - RELATIONSHIP BETWEEN THE RELATIVE WAVE PERIOD, $T_D/T_F$, OF $F_{min}$ = 200 NAUTICAL MILES AND $F_{min}$ = 50 TO 1000 NAUTICAL MILES
EXAMPLE: $T_F$ = 10 secs, D = 2000 nautical miles, $F_m$ = 400 nautical miles, $T_D/T_F$ = 1.28, $T_D$ = 1.28 x 10 = 12.8 seconds
C - RELATIVE DECREASE OF WAVE HEIGHT, $H_D/H_F$, WITH RESPECT TO DECAY DISTANCE FOR VALUES OF $H_F$ AT END OF $F_{min}$ = 200 NAUT MILES
D - RELATIONSHIP BETWEEN THE RELATIVE WAVE HEIGHT, $H_D/H_F$, OF $F_{min}$ = 200 NAUTICAL MILES AND $F_{min}$ = 50 TO 1000 NAUTICAL MILES
EXAMPLE: $H_F$ = 13 feet, D = 2000 nautical miles, $F_m$ = 400 nautical miles, $H_D/H_F$ = .251, $H_D$ = 13 x 251 = 3.26 feet
$T_F$, SECONDS, SIGNIFICANT WAVE PERIOD AT END OF $F_{min}$
$T_D/T_F$, RELATIVE WAVE PERIOD AT END OF DECAY DISTANCE
$T_D/T_F$, RELATIVE WAVE PERIOD — FOR $F_{min}$ = 200 NAUTICAL MILES
DECAY DISTANCE, D, IN NAUTICAL MILES
$T_F$ - Seconds at end of $F_{min}$ = 200 Nautical Miles
$H_F$ - Feet at end of $F_{min}$ = 200 Nautical Miles
$H_D/H_F$, RELATIVE WAVE HEIGHT — FOR $F_{min}$ = 200 NAUTICAL MILES
$H_D/H_F$, RELATIVE WAVE HEIGHT AT END OF DECAY DISTANCE
$H_F$, FEET, SIGNIFICANT WAVE HEIGHT AT END OF $F_{min}$
EXAMPLE
UNIVERSITY OF CALIFORNIA HYO-5430-155

FIGURE VI A-8

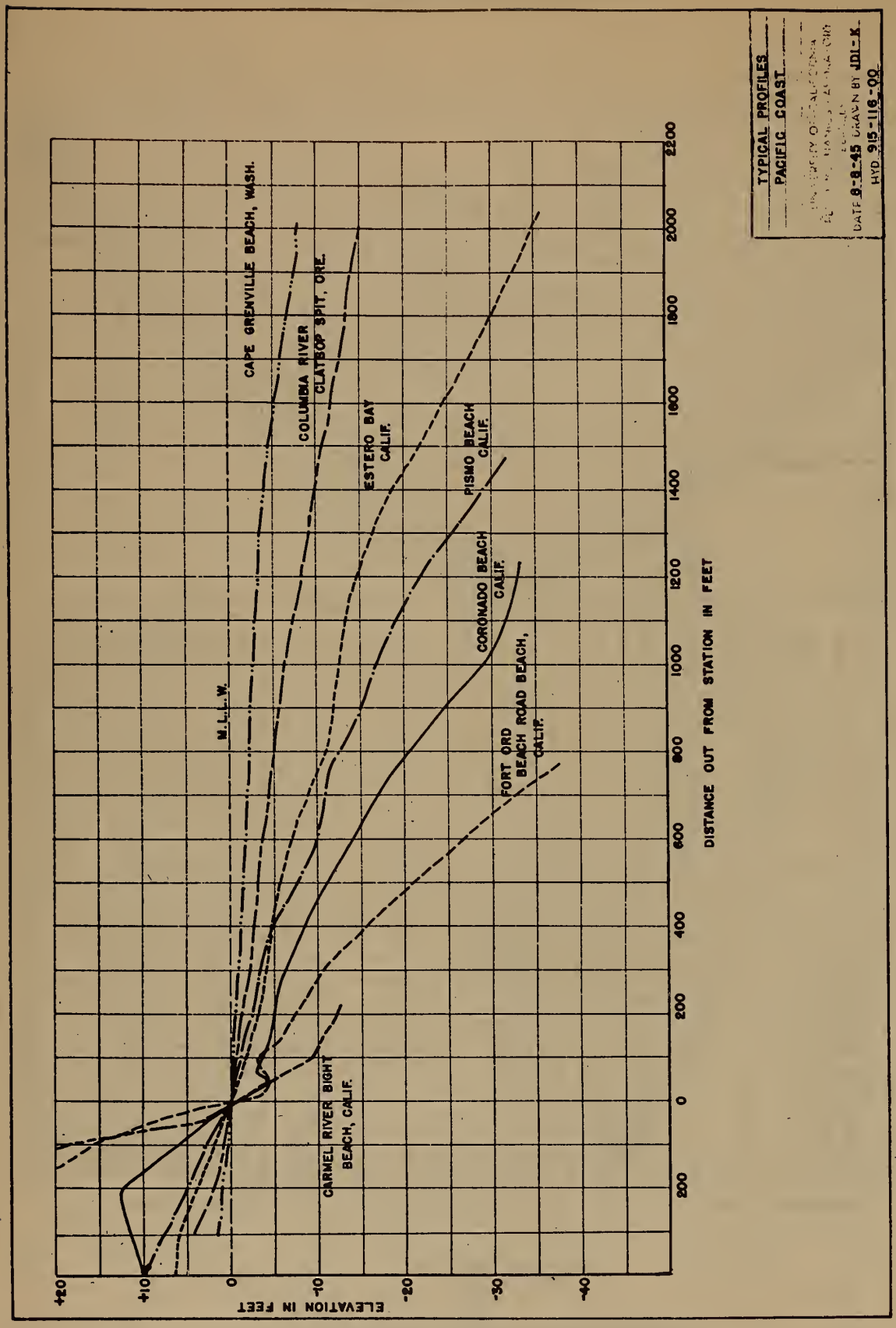

TYPICAL PROFILES
PACIFIC COAST
DATE 9-8-45 DRAWN BY JDI-K
HYD. 915-116-99
CAPE GRENVILLE BEACH, WASH.
COLUMBIA RIVER
CLATSOP SPIT, ORE.
ESTERO BAY
CALIF.
PISMO BEACH
CALIF.
CORONADO BEACH
CALIF.
FORT ORD
BEACH ROAD BEACH,
CALIF.
CARMEL RIVER BIGHT
BEACH, CALIF.
M.L.L.W.
DISTANCE OUT FROM STATION IN FEET
2200
2000
1800
1600
1400
1200
1000
800
600
400
200
0
200
ELEVATION IN FEET
+20
+10
0
-10
-20
-30
-40

FIGURE VIA-9

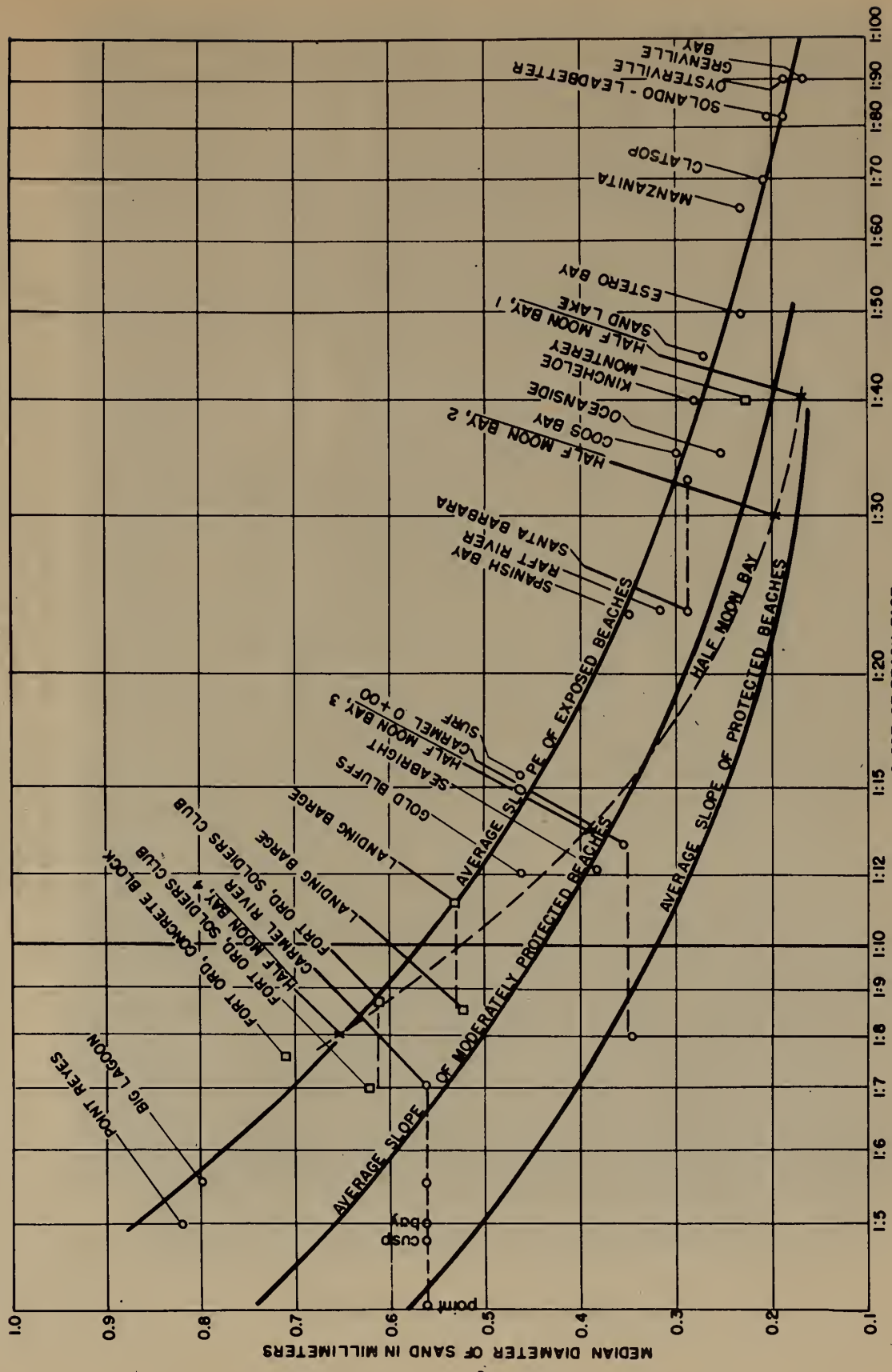



FIGURE XIA-10 - SIZE-SLOPE RELATIONSHIP AT THE REFERENCE POINT

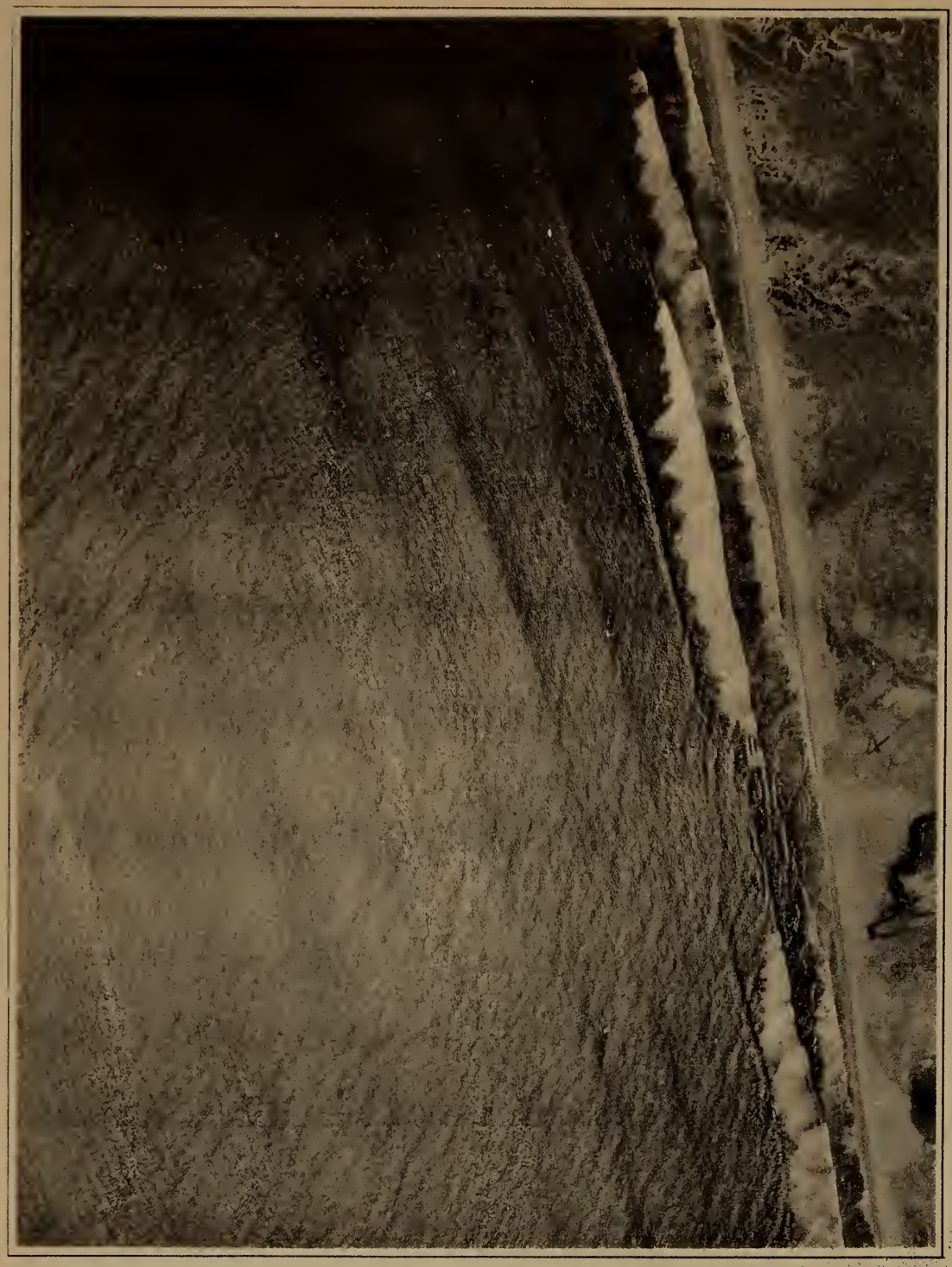
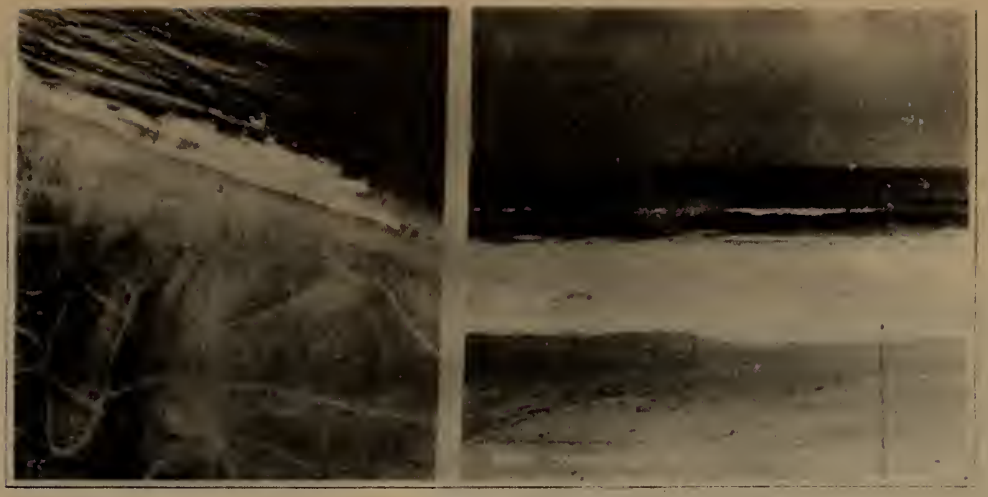
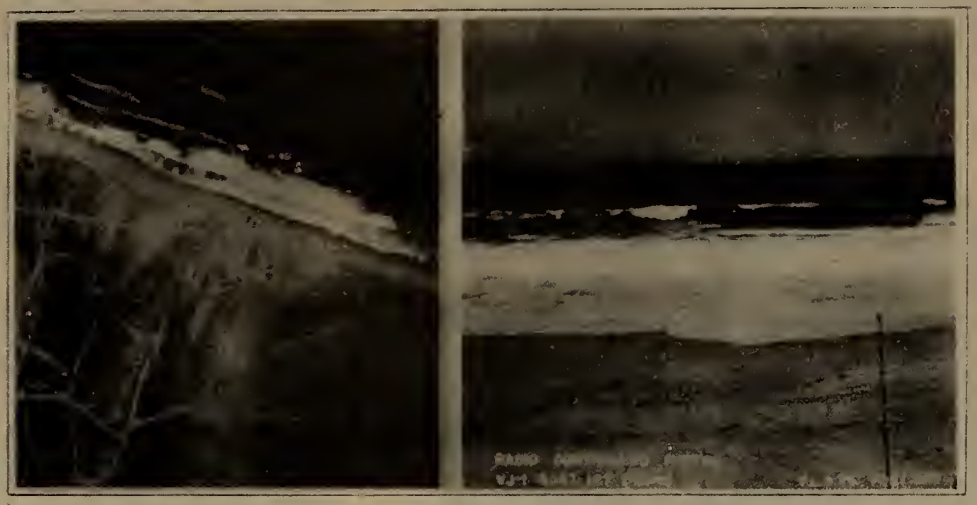


Photo No. 5528. Waves breaking at an angle with the shore of Aliso Canyon near Oceanside. Period = 18 sec. Height of breaker = 6.5 feet. Depth of breaking = 5 ft.

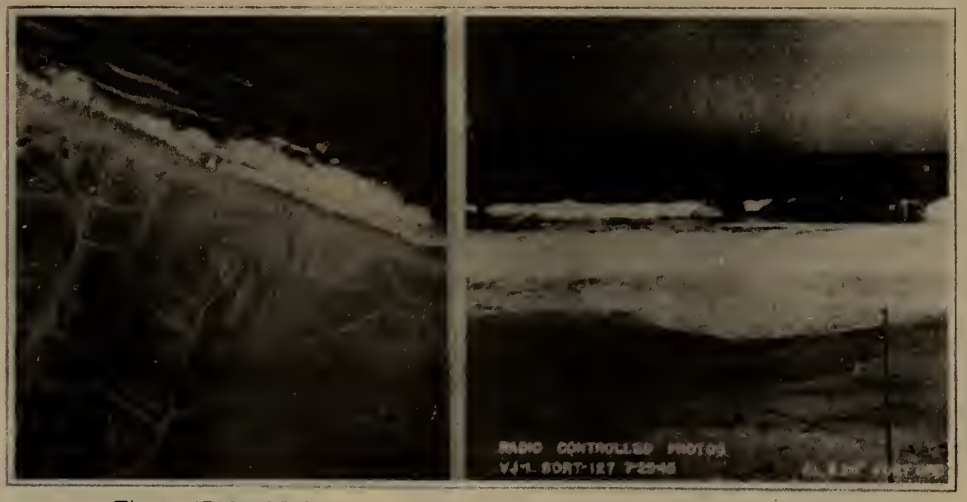
FIGURE XI A-11



a. Radio controlled surf pictures, Fort Ord, 7/25/45. Time: 1310 23.00



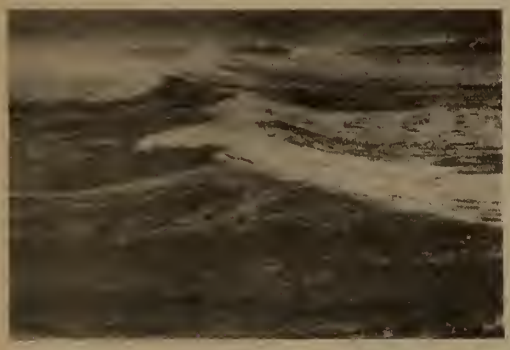
b. Time: 1310 24.00



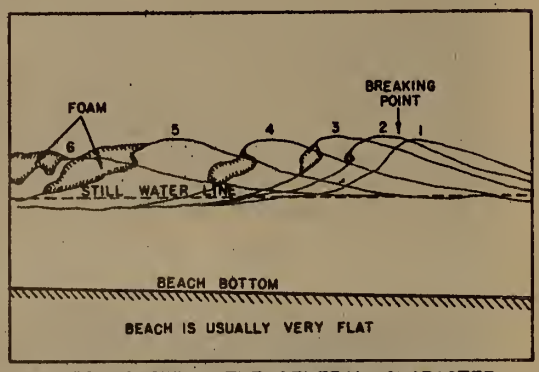
c. Time: 1310 25.00

FIGURE XI A-12

Both photographs and diagrams of the three types of breakers are presented below. The sketches consist of a series of profiles of the wave form as it appears before breaking, during the breaking, and after breaking. The numbers opposite the profile lines indicate the relative times of the occurrences.



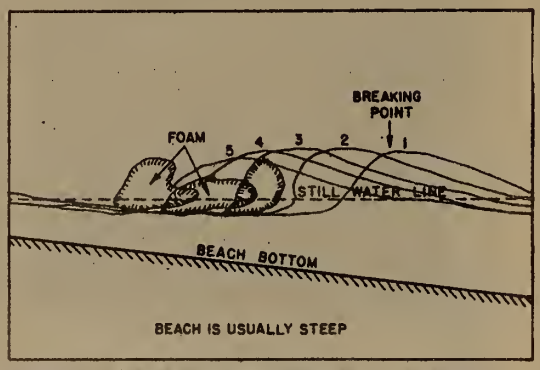
SPILLING BREAKER




SKETCH SHOWING THE GENERAL CHARACTER OF SPILLING BREAKERS



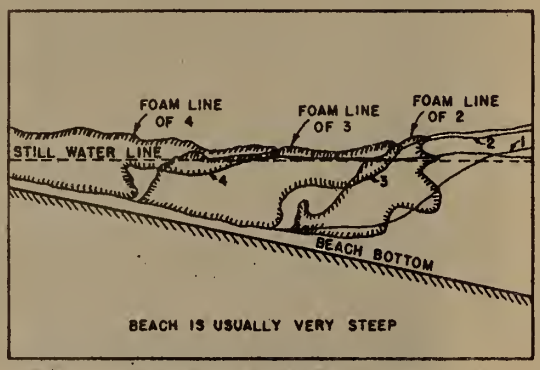
PLUNGING BREAKER




SKETCH SHOWING THE GENERAL CHARACTER OF PLUNGING BREAKERS



SURGING BREAKER




SKETCH SHOWING THE GENERAL CHARACTER OF SURGING BREAKERS

FIGURE VIA-13 - BREAKER TYPES



a. Plunging breaker. San Simeon, California, 4/8/45
$H_b$ = 3.5 feet, T = 17 seconds



b. Irregular plunging breaker. San Simeon, 4/8/45
$H_b$ = 4 feet, T = 17 seconds



c. Wind wave breaking in deep water. Estero Bay, 5/26/45
$H_b$ = 4 feet, T = 6 seconds



d. Spilling breakers on a very irregular beach with steep-faced broken bars. Pt. Sur, 7/27/45
$H_b$ = 8 feet, T = 9 seconds

FIGURE VII A-14



RESTRICTED Security Information



a. Swell peaking up. Estero Bay, 6/14/45
$H_b$ = 7 feet, T = 14 seconds



b. Wind waves and swell breaking on a regular beach. Estero Bay
Wind wave: $H_b$ = 4 feet, T = 6 seconds
Swell: $H_b$ = 4 feet, T = 14 seconds



c. Pt. Joe Bight, 6/10/45



b. Pt. Cypress Bight, 6/10/45

Two bights of identical exposure to the sea, taken in the same hour. One difference is due to refraction.

FIGURE VI A-15



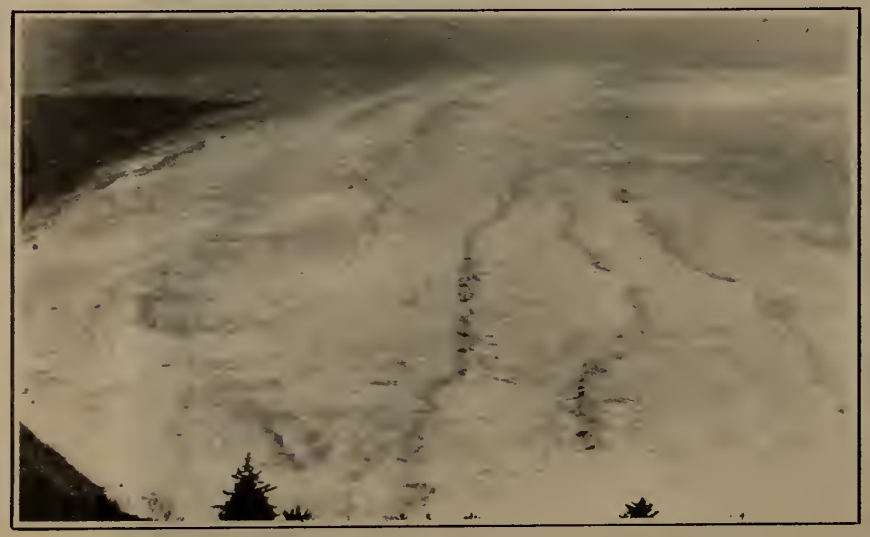
RESTRICTED Security Information



a. Waves breaking at an angle to the beach.
from SW: $H_b$ = 6 feet, T = 19 seconds
" WNW: $H_b$ = 4 feet, T = 9 seconds



b. Oblique breaker from wind wave superimposed upon swell. Note offshore swell. Estero Bay, 5/17/45
Swell: $H_b$ = 6 ft, T = 16 secs; Wind waves: $H_b$ = 4 ft, T = 7 secs



c. Large storm waves arriving. Manzanita Beach, 11/1/46
$H_{b(max)}$ = 18 feet, T = 13.3 seconds



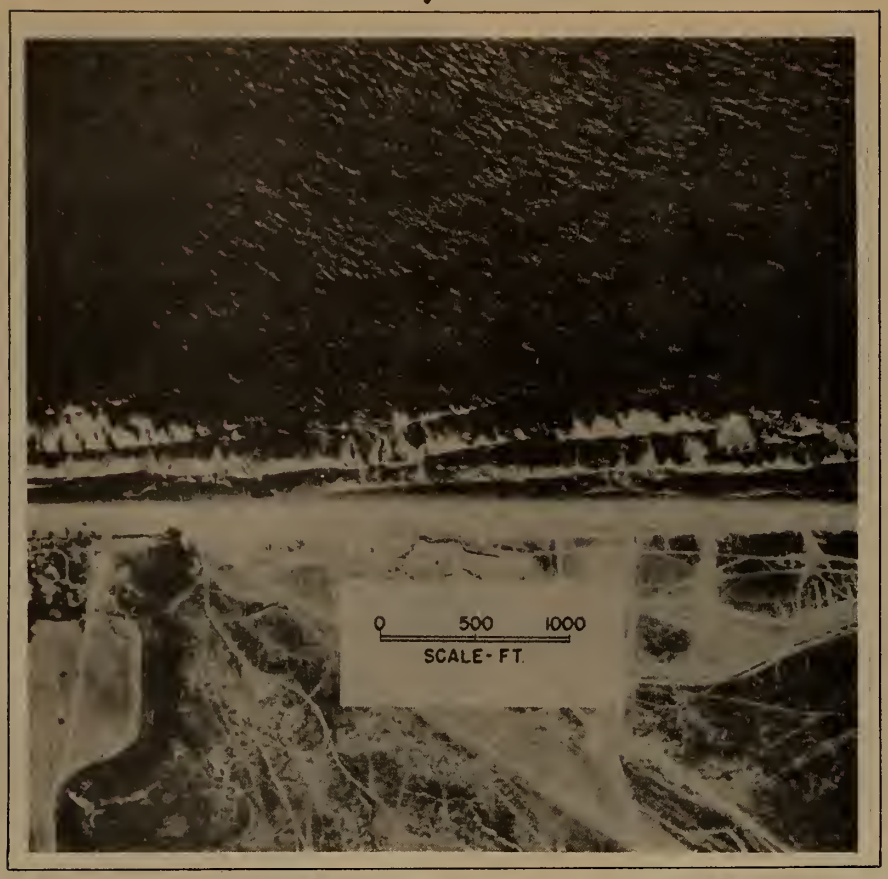
d. Storm waves start to diminish. 11/5/46
$H_{b(max)}$ = 12 - 14 feet, T = 13.1 seconds

FIGURE VI A-16



RESTRICTED Security Information

CLASSIFICATION CANCELLED OR CHANGED TO Confidential BY AUTHORITY OF OPNAV ltr 26034 8 Dec 53
(DATE) (SIGNATURE) M. H. Brown (RANK) Asst.




Aerial photograph of surf at Oceanside, California. Two wave systems are present in this picture. There are small wind waves coming from the upper right, and a long, low swell from the upper left. The swell is almost invisible in deep water, but peaks up near the shore to form the predominate breakers. Note how the waves break in short segments, where crests of the wind waves are superimposed on the crest of the swell. The wind waves have a deep water wave length of about 50 feet and a height of 1-2 feet. The swell has a deep water wave length of about 1000 feet, with a height of 2-3 feet. The breaker height is about 5 feet.



Long period swell almost obliterated but still controls line of breaking. Angularity of superimposed wind waves results in spilling. Estero Bay, 5/17/45
$H_b$ = 6 feet, T = 16 secs.
$H_b$ = 4 feet, T = 7 secs.

FIGURE VIIA-17



FIGURE VII A-18 - Refraction at HALF MOON BAY. Waves breaking up going over reef. 11/21/44.
T (inside) = 16 seconds, T (outside) = 20 seconds

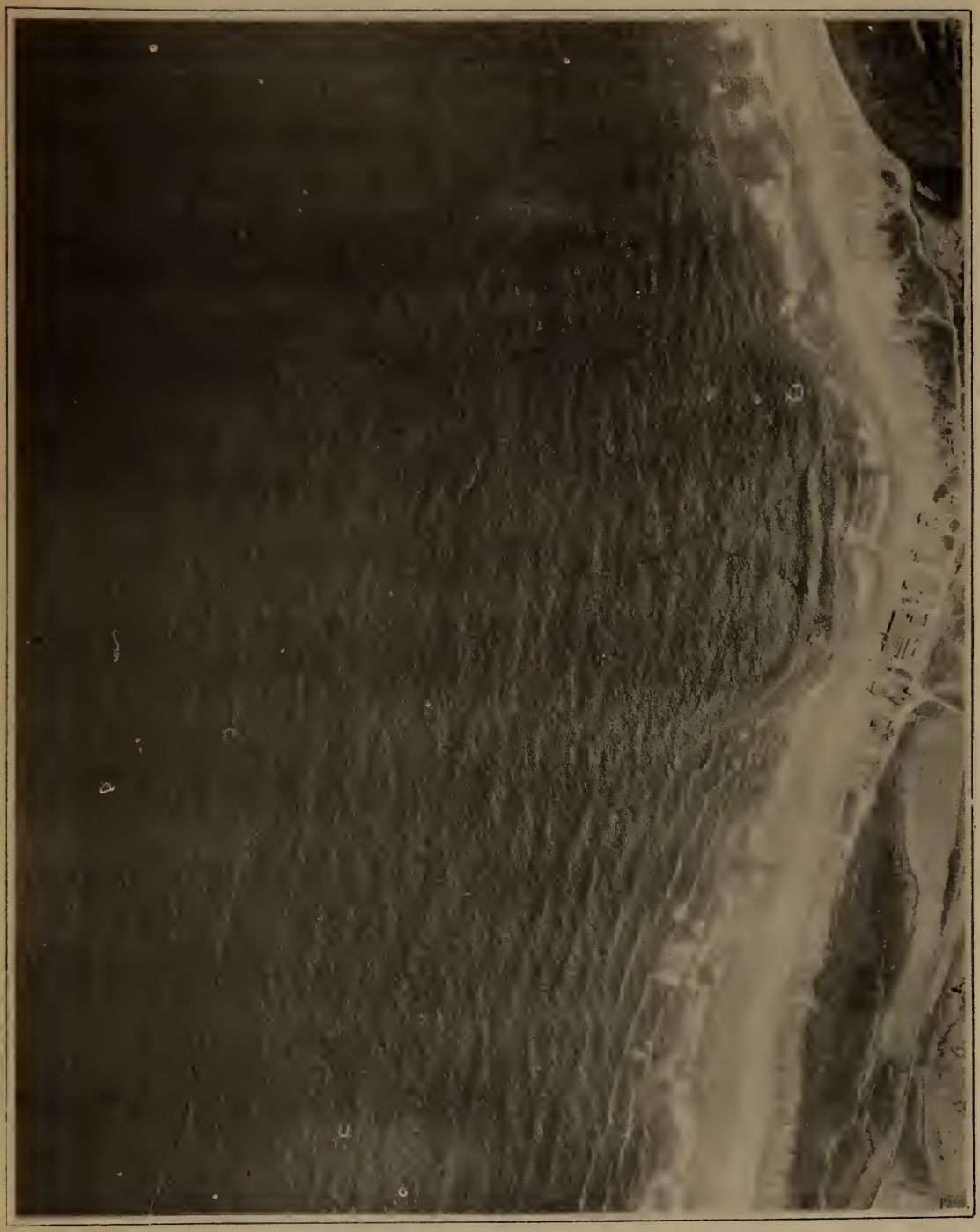


Photo No. 5527. Effect of submarine canyon on wave front. Moss Landing. 6/16/43. $H_b$ = 4.0 feet. T = 8.5 seconds.

FIGURE XII A-19

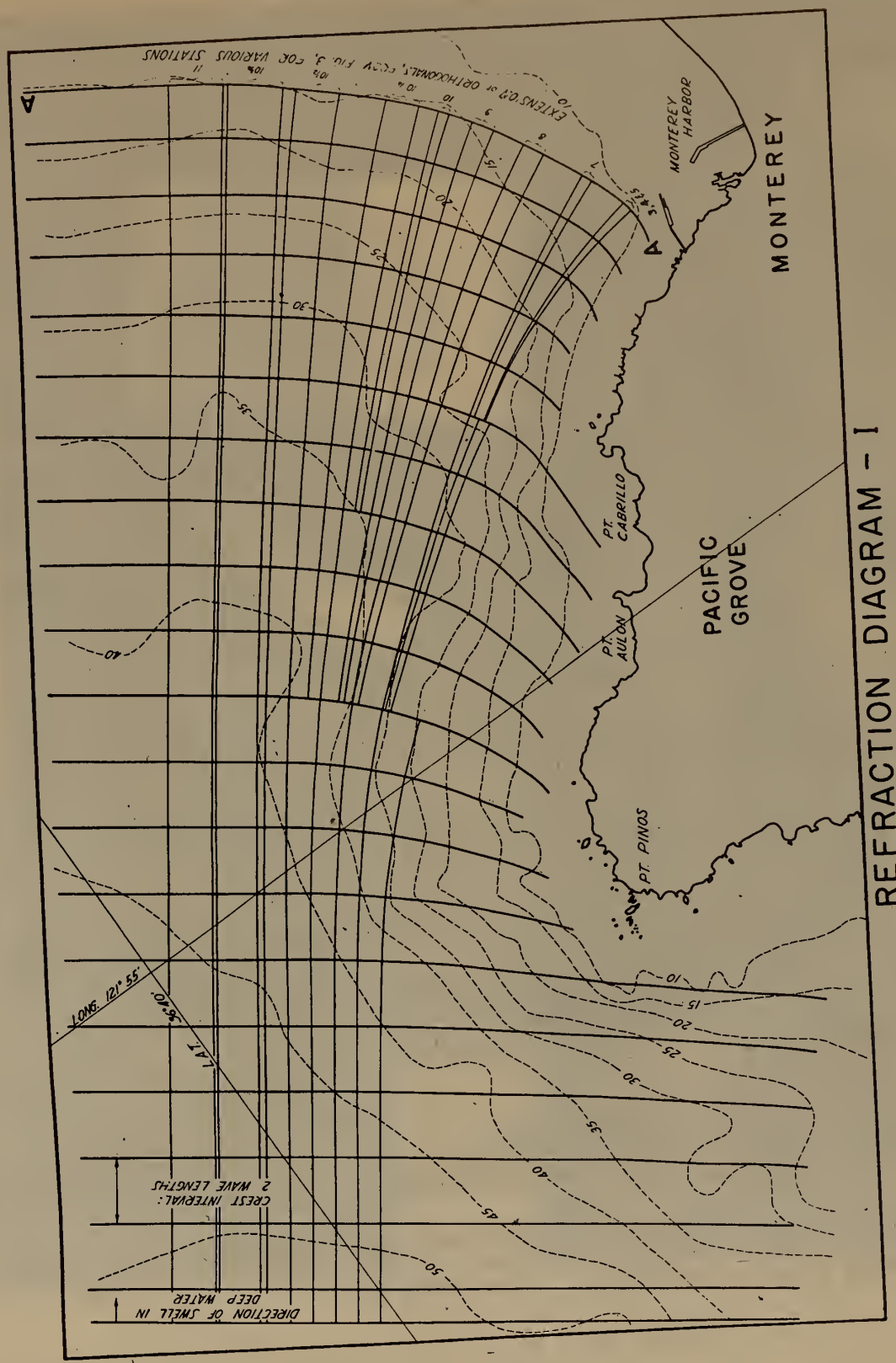


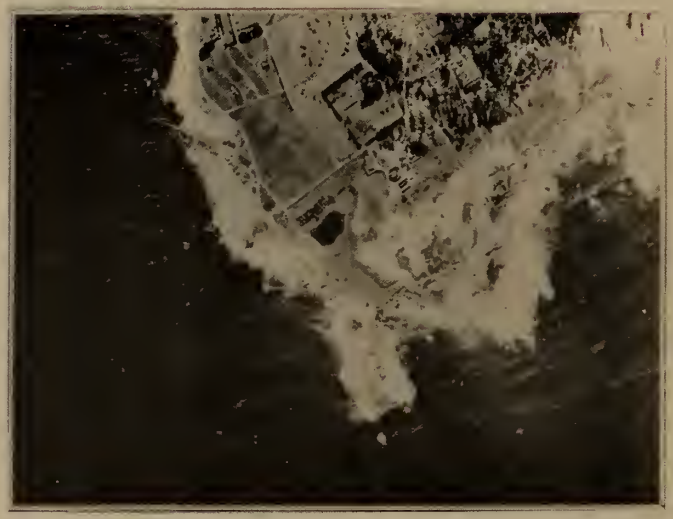



FIGURE VII A-20




Photo No. 5539. Mosaic of Monterey Harbor. Refraction study. Variation of surf zone around the harbor and showing effect of breakwater. Date - 2/24/45.

FIGURE VI A-21



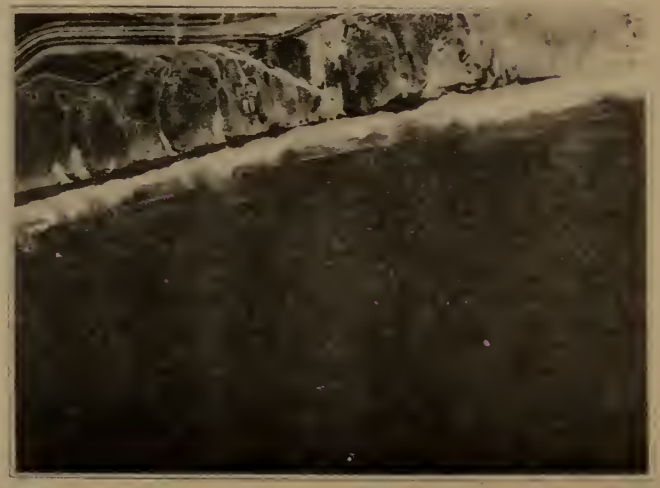
a. Wave action off Point Pinos, Monterey Harbor. Wave direction 306°.
H = 9 feet
$T_{(ave)}$ = 9.8 seconds



b. Wave action behind Monterey Breakwater.
$H_{(ave)}$ = 0.7 - 0.9 feet
T = 12.7 seconds



c. Monterey Harbor refraction, covering Stations 5 to 10. Breaker heights range from 0.6 to 11.4 feet.
$H_o$ = 9.0 feet
T = 12.7 seconds



d. Wave and surf action near Station 11. Breaker height 11.4 feet.
H = 8.4 feet
T = 12.7 seconds

FIGURE VI A-22



RESTRICTED Security Information

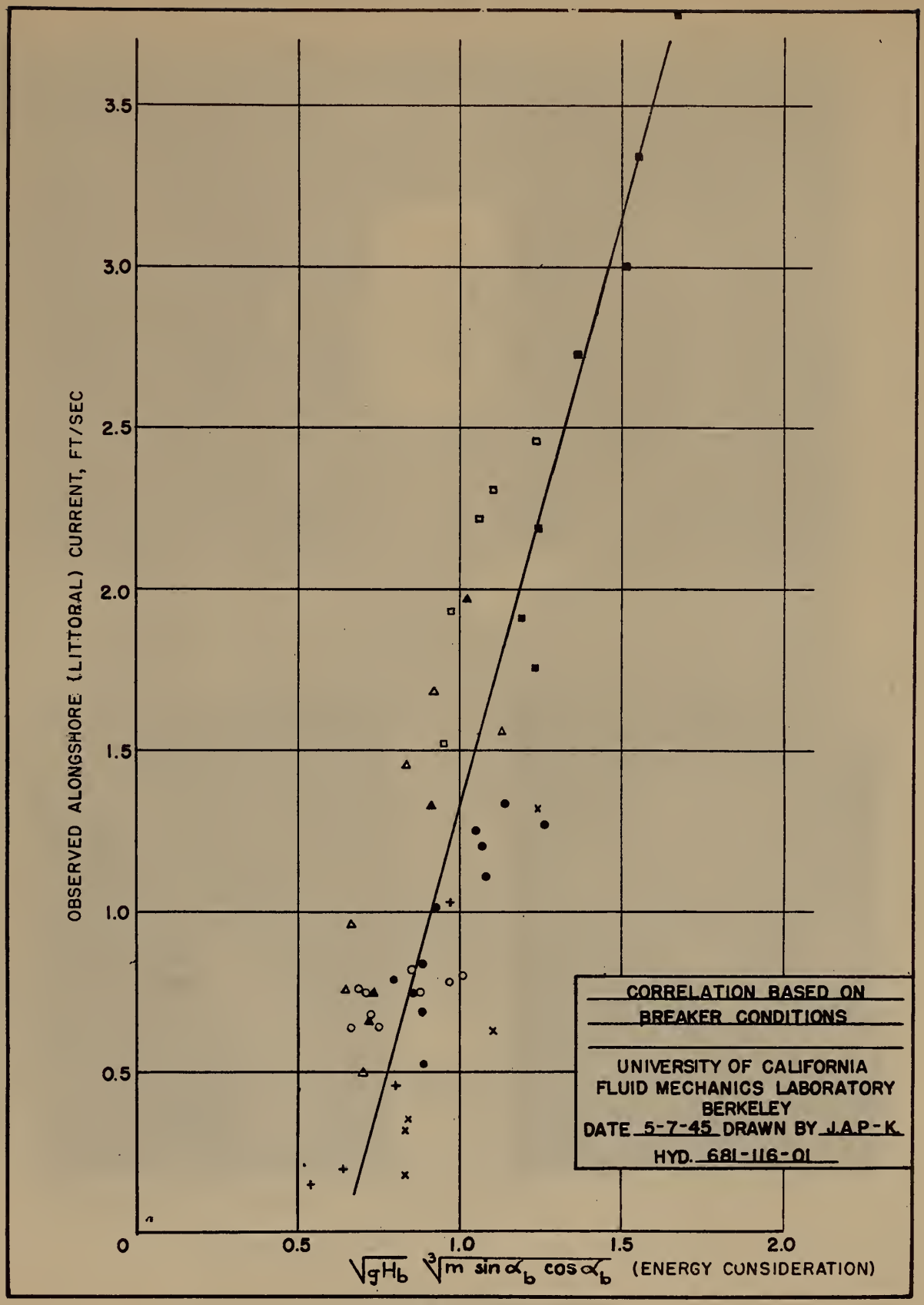



FIGURE VIA-23

# MANUAL OF AMPHIBIOUS OCEANOGRAPHY

## SECTION VI - INTELLIGENCE FOR AMPHIBIOUS OPERATIONS

### B. WAVE, SURF AND BEACH INTELLIGENCE FOR AMPHIBIOUS OPERATIONS

BY M. P. O'BRIEN, R. L. WIEGEL, J. W. JOHNSON, W. N. BASCOM, D. K. TODD

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## B. WAVE, SURF AND BEACH INTELLIGENCE FOR AMPHIBIOUS OPERATIONS

### 1. Objectives

In military operations involving amphibious warfare there are two general classifications of intelligence-strategic and tactical.

Strategic Intelligence probably corresponds to the "pre-planning tasks" of Naval Intelligence. It includes the collection, evaluation and distribution of information which is necessary in the selection of objectives. Information must be obtained and evaluated so that questions of the following types may be answered:

(i) Can landings be made in sufficient force to establish a beach-head? (Size and type of shorelines, terrain, tides, wave climate, etc.)

(ii) Can sufficient material be supplied to forces on shore? (Harbor facilities, possibility of artificial harbors, feasibility of supplying extended operations across beaches, etc.)

Tactical Intelligence corresponds to the "Planning" and "Execution" phase of Naval Intelligence. After the objectives have been set forth, detailed information must be obtained and evaluated for use in planning and performing the operation.

The notes presented herein are intended to outline practical means of obtaining the maximum information regarding an enemy-held shoreline.

a. Surf Zone

(i) Size, type, and period of breakers.

(ii) Description of surf zone.

(iii) Range of tides.

(iv) Number, depth and locations of offshore bars. Channels through bars.

(v) Longshore currents (magnitude and direction).

(vi) Rips, location, size and magnitude of currents.

(vii) Defenses (traps, mines, etc.).

(viii) Composition of bottom.

(ix) Nature of rocks and reefs.

(x) General hydrography.

(xi) Profile of beach through surf zone.

b. Beach Zone

(i) Slope of beach face. Slope of upper beach.

(ii) Composition of the beach material.

(iii) Trafficability of the beach from water line to berm. Across berm.

(iv) Exits from the beach. Location, numbers, types and trafficability.

(v) Beach defenses (traps, mines, etc.).

(vi) Debris on beach.

(vii) Protective cover after landing.

(viii) Size and type of beach.

c. Back Beach

(i) Nature of the beach-backbeach contact (dunes, cliff, etc.).

(ii) Kind and size of vegetation.

(iii) Roads available and road building equipment needed.

(iv) Nature of likely defenses.

(v) Storage areas.

(vi) Relation to population centers.

(vii) Potable water.

The ability to perform each item in the outline successfully is dependent on the degree of cooperation and coordination between aerologists, wave forecasters, photoplane operations officer, U.D.T. (underwater Demolition Team) reconnaissance team officer, engineering officer (tide calculations and refraction diagrams), photographic interpreters, and all the photographic personnel (pilots, photographers, lab technicians, etc.). All of these personnel should have an understanding of the requirements to be met so that they are alert to anomalies and are better able to exercise initiative. Expensive and risky sorties by photoplanes, U.D. Teams, and reconnaissance teams can be reduced to a minimum be planning conferences in which all of these items are considered. Reconnaissance flights should not be sent out when conditions are not favorable; good conditions must not be missed.

2. Published Sources of Information

This section lists, and describes briefly, the more significant of the many sources of information for use by intelligence staffs.

a. Charts and Maps

(1) Topographic Maps: Topographic maps are available in many countries of the world. Some of the U. S. agencies from which they may be obtained, are as follows:

(a) U. S. Army - Corps of Engineers: Tactical maps of many areas.

(b) Interior Department - Geological Survey: Topographical and geologic maps.

(c) U. S. Navy - Hydrographic Office: Charts of foreign countries. An example of a chart for the Chiniak Bay region (Misc. #11,871-4) is shown in Figure VI B-1 (to a reduced scale).

(2) Nautical Charts: Nautical charts are available for most parts of the world. They are published by the Coast and Geodetic Survey (U. S. Department of Commerce) in the United States and by the British Admiralty in Great Britain. Nautical charts also have been compiled by the Japanese.

The U. S. Coast and Geodetic Charts are classified as follows:*

(i) "Sailing Charts: Scales 1:600,000 and smaller. These are planned for use in fixing the mariner's position as he approaches the coast from open ocean, or for sailing between distant, coastwise ports. On such charts the shoreline and topography are generalized and only offshore soundings, the principal lights, outer buoys, and landmarks visible at considerable distances are shown. (Figure VI B-2 is a reproduction of a section of Chart 5020.)

(ii) General Charts: Scale 1:100,000 to 1:600,000. These are planned for coastwise navigation outside of outlying reefs and shoals. (Figure VI B-3 is a reproduction of a section of chart 6002.)

(iii) Coast Charts: Scale 1:50,000 to 1:100,000. These are planned for inshore navigation; for entering bays and harbors of considerable width; and for navigating large inland waterways.

(iv) Harbor Charts: Scales larger than 1:50,000. These are planned for harbors, anchorage areas, and the smaller waterways." (Figure VI B-4 is a reproduction of a section of Chart 6185.)

(3) Field Surveys: "Topographic and hydrographical surveys, the principal source of chart material, are plotted on larger scales than those on which the charts are published. They provide a comprehensive record of conditions existing at the specific time the survey was made, while the chart is generally a compilation of data secured at various times. When information is desired on a larger scale than that of the chart, or where the actual date applicable to the information shown is important, the public frequently is better served by having recourse to the field surveys, photographic copies of which may be obtained. . . ."**

Figure VI B-5 shows a reproduction of a part of one of these work sheets.

(4) Sea and Swell Charts: Published by the U. S. Navy Hydrographic Office. These charts, for each month of the year, were compiled from observations taken during the period of 1932-1942. They show the direction and magnitude of sea and swell, together with the percentage of time. Figure VI B-6 is a reproduction of a section of H.O. Misc. No. 10,712-G-1.

(5) Current Charts: Published by the U. S. Navy Hydrographic Office. Information relating to monthly surface currents are shown on these charts. (Figure VI B-7 is a reproduction of a section of H. O. Misc. 10,057-1.

*Quoted from "Catalogs of Nautical Charts and Related Publications", U. S. Coast and Geodetic Survey, Serial Number 665, July 1946. (Reference VI B-1)

**Quoted from Reference VI B-1.

b. Souces of Meteorological Data and Information:

The following listing includes descriptions and sources of weather maps and meteorological publications currently available. For each item a brief description is first given, followed by the sources in parentheses. In the second section, however, the source is not indicated since all of these publications are available from the Superintendent of Documents, Government Printing Office, Washington 25, D. C. Most of these publications are also available in depository libraries throughout the United States.

For synoptic weather maps prepared by the Armed Forces, local inquiry will be necessary to determine which maps are available. Almost as many maps are available at smaller bases as at larger bases through the use of military facsimile transmissions. The same is also true for prognostic maps.

(1) Weather Maps

(a) Historical--Northern Hemisphere:

(i) Daily synoptic series, historical weather maps, Northern Hemisphere, sea level--486 volumes, bound by months, containing analyzed daily weather maps, 13 inches square, showing data and weather analysis for 1300 GMT. They cover the period from January 1899 to June 1939. (Sup't. of Documents)

(ii) Same as above, for period October 1945 to present time. (U. S. Weather Bureau)

(b) Historical--Southwest Pacific:

Daily synoptic series, historical weather maps, Southwest Pacific, sea level--36 volumes, 8 by 13 inches, bound by months, containing analyzed daily weather maps of the Southwest Pacific area extending from 70° E., longitude, to 145° W., longitude, and from 30° N., latitude, to 50° S., latitude, for 0700 130th meridian time. They cover the years 1932 to 1934, inclusive. (Sup't. of Documents)

(c) Synoptic:

(i) Northern Hemisphere, daily synoptic series, unbound, ozalid prints. (U. S. Weather Bureau and principal domestic forecasting centers)

(ii) Southern Hemisphere, daily synoptic series, recently available in microfilm rolls, about 3 months after map date. (U. S. Weather Bureau)

(iii) Daily weather map, size 19 by 24 inches, issued at Washington, D. C., shows 1:30 a.m. surface weather data for the U. S., the barometric and frontal pattern over North America as it existed

12 hours previously, precipitation totals for the 24 hours previous to the map time, highest and lowest temperatures for the previous 24 hours, and the 700-millibar constant pressure chart.
(Sup't. of Documents)

(iv) Six-hourly sea level weather maps, and 12-hourly 700-mb and 500-mb constant pressure charts, areas covered depend on location of base, unpublished, unbound.
(U. S. Air Force and U. S. Navy Bases) (Figure VI A-1)

(d) Prognostic:

Twelve-hourly sea level and 700-mb prognostic weather maps, areas covered depend on location of base, unpublished, unbound.
(U.S. Air Force and U. S. Navy Bases)

(2) Meteorological Publications (all available from Sup't of Documents):

(i) Atlas of climatic charts of oceans, 1938, 130 charts (Figure VI B-8 shows an example of the type of information available).

(ii) Average conditions of wind and weather North Atlantic Ocean and West Indian waters, 1936, 9 p. North Pacific Ocean, 1937, 12 p., illustrated.

(iii) Average monthly weather, a semimonthly publication, presenting a resumé of the past month's weather and an outlook for the coming month.

(iv) Classified flying weather for the United States, percentage frequencies of contact, instrument, and closed weather, 1946, 300 p. illustrated.

(v) Climatological data
Alaska, issued monthly, with annual summary.
Hawaii, issued monthly, with annual summary.
West Indies, issued monthly, with annual summary.

(vi) Instructions to marine meteorological observers, 7th ed., 1941, 114 p. illustrated.

(vii) International weather station numbers for North and Central America, effective 0030 GCT. January 1, 1949, 1948, 44p.

(viii) Normal weather maps, Northern Hemisphere
Sea level pressure, 1946, 1p., illustrated 13 maps.
Upper level, 10,000 feet (3050 meters) and 20,000 feet (6100 meters), 10, 13, 16, and 19 kilometers, 1944. 2 p., illustrated, 72 maps.

(ix) Preparation and use of weather maps at sea, 3rd ed., 1949. 117 p., illustrated map.

(x) Storm warning manual, 1946, 16p.

(xi) Weather broadcasts--directory of weather broadcasts made by Weather Bureau personnel from microphones installed in Weather Bureau offices, 1949, 11 p.

(xii) Wind frequency distribution. North Atlantic Ocean, 6 and 10 kilometers, 1946. 52 p., illustrated. Northwest Pacific Ocean, 3, 6, and 10 kilometers, 1945, 1p., 18 maps.

c. Tidal Publications:

(1) Tide Tables: Published by the Coast and Geodetic Survey, (U. S. Department of Commerce), the British Admiralty and the Japanese.

The U. S. Tide Tables furnishes "advance information relative to the rise and fall of the tide in annual tide tables. These tables include the predicted times and heights of high and low water for every day in the year for a number of reference stations and differences for obtaining similar predictions for numerous other places. In addition they include other useful information such as methods for obtaining heights of tide at any time, local civil time of sunrise and sunset for various latitudes, reduction of local civil time to standard time of moonrise and moonset for certain ports, and astronomical data."*

Figure VI B-9 is a reproduction of one page of a tide table.

(2) Current Tables: The current tables, published by the U. S. Department of Commerce, Coast and Geodetic Survey, "cover the predicted time of slack and the times and velocities of strength of current, and include tables on duration of slack, coastal tidal currents, wind currents, combination of currents, and current diagrams of certain waterways."* Figure VI B-10 is a reproduction of a page from "Current Tables, Pacific Coast, 1948".

d. Miscellaneous Nautical Publications:

(1) United States:

(a) U. S. Coast Pilots: "Each book furnishes information required by the navigator for the area indicated, that cannot be shown conveniently on the nautical chart such as detailed data relative to the coast line, port information, sailing directions for coasting and entering harbors, and general information as to weather conditions, radio service, etc. New editions are published about every seven years."*

The volumes available at present (1951) are Atlantic Coast, Gulf Coast, Intracoastal Waterway (Inside Route), West Indies, Pacific Coast, Hawaiian Islands, Philippine Islands, and Distance Tables.

(b) Notice to Mariners: "A weekly leaflet issued by the U. S. Coast Guard, Treasury Department, . . . . Notice to mariners is issued as a safety aid, to keep mariners advised of constant changes so they may keep their nautical charts, and new publications are also announced therein."*

* Quoted from Reference VI B-1.

(c) Sailing Directions: These are published by the U. S. Navy Hydrographic Office for most parts of the world. They include much detailed and general information of the various regions. As an example, the following is quoted from H. O. No. 135 (Sailing Directions for the North Sea, Eastern Shores Section A, 1933, pp. 250).

"1642. Depths.--On Havrevlen, the bar of Thyboron Channel, there are depths of 16 to 19 feet (4.9 to 5.5 m) but the bar changes frequently. At present the controlling depth of Thyboron Harbor is the depth over Havrevlen. There is a heavy surf over the bar during strong westerly winds. Inside Havrevlen the depths in the channel range from 18 to 25 feet (5.5 to 8.5 m)."

(2) Great Britain: The following are some of the publications available from the British Admiralty:

(i) Catalogue of Admiralty Charts and other Hydrographic Publications.

(ii) Ice charts of the Northern Hemisphere.

(iii) Ice chart of the Baltic and German North Sea Coasts.

(iv) Prevailing Oceanic Winds.

(v) Baltic Pilot, Volumes I, II and III.

(vi) Artic Pilot, Volumes, I, II and III.

(vii) Black Sea Pilot.

(3) International Hydrographic Bureau: The catalog as well as publications may be obtained from the International Hydrographic Bureau, Monte Carlo, Monaco (Principality).

e. Operational Reports:

General action reports have been prepared for all the amphibious operations carried out by the U. S. Navy during World War II. These reports are comprised of narrations of the operations, together with comments on various technical matters. It might be possible to obtain information on beaches which are similar to the one under study as well as to learn pertinent operation methods.

For example, in the "Report on the Capture of Iwo Jima" (Commander Task Force Fifty-One, Commander Amphibious Forces, U. S. Pacific Fleet, CONFIDENTIAL) the following statements are made:

"The steep gradient and the soft volcanic cinders of the EASTERN Beaches enabled a very moderate surf to present a continuous hazard to the beaching and unloading of all types of boats. Boats which could normally beach were broached, and the resultant wave action quickly shifted the soft sand, making salvage most difficult. Beach conditions were not favorable for the movement of vehicles and only limited amounts of critical supplies were landed. Beaches BLUE and YELLOW were closed at 2311 due to heavy gunfire and all traffic was concentrated on RED and GREEN Beaches."

"Landing boats were particularly ineffective during this operation due to the nature of the beaches. The slope at the water's edge was very steep, the

black volcanic sand was soft, loose, and more nearly resembled a very light fine cinder which would not pack, then a sand beach."

"LVTs were particularly useful in this operation due to the difficulties encountered in operating wheeled vehicles in the soft sand of the beaches. They were used in unloading from the LSTs during the early stages of the assault and moved their loads to inland dumps as to provide direct troop support."

f. Wave Records:

For the past several years wave recorders have been in operation at a few locations such as Pendeen, England; Guam, M. I.; Cuttyhunk, Massachusetts; Banana River, Florida; Pt. Sur, Pt. Arguello, Camp Pendleton and La Jolla, California; Heceta Head, Oregon. It is believed that the French also have some recorders operating in France. At the present time, however, no agency is assembling the data from the various sources.

g. Ship Logs:

Much data could be obtained from old ship logs.

h. Photographs:

The various services have accumulated many thousands of photographs (aerial, sea and ground) for various parts of the world.

i. Books:

There are many books on oceanography, geology, geography, etc. available from which valuable information could be obtained, such as:

(i) "The Oceans" by Sverdrup, Johnson and Fleming.

(ii) "Oceanography for Meteorologists" by H. U. Sverdrup.

(iii) "The U. S. Marines and Amphibious War" by Isely and Crowl.

(iv) "Shoreline Processes and Shoreline Development" by D. W. Johnson.

(v) "Coasts, Waves and Weather" by John Q. Stewart.

(vi) "The Sea Around Us" by Rachael Carson.

(vii) "Wave Action in Relation to Engineering Structures" by D. D. Gaillard.

j. Papers:

Numerous papers have been written on various phases of wave action and on many beaches. Some of the publications in which these papers may be found are:

(i) Accounts of the Permanent International Association of Navigation Congresses.

(ii) Transactions of the American Geophysical Union.

(iii) Bulletin of the Beach Erosion Board (U. S. Army - Corps of Engineers).

(iv) Bulletin D'Information, Comite Central D'Oceanographie et D'Etudes Des Cotes (Services Central Hydrographique - 13, Rue de l'University, Paris, France).

(v) Bulletin of the Geological Society of America.

(vi) Journal of Sedimentary Petrology.

(vii) Journal of Geology (University of Chicago)

(viii) Journal of the Institution of Civil Engineers (England)

(ix) Proceedings of the Geologists Association (London, England)

(x) Quarterly Journal of the Geological Socity of (London, England)

(xi) Nature (England)

k. Reports:

There are many reports available at institutions which are doing research in related subjects. Some of these agencies are:

(i) Institute of Engineering Research, University of California.

(ii) Scripps Institution of Oceanography, (La Jolla, California)

(iii) Beach Erosion Board (U. S. Army, Corps of Engineers.)

(iv) Woods Hole Oceanographic Institution.

(v) U. S. National Bureau of Standards.

(vi) Cornell University.

(vii) Texas A. & M.

(viii) National Institution of Oceanography (London, England)

(ix) Imperial College (London, England)

(x) Trinity College, Cambridge University (Cambridge, England)

(xi) Etr. Neyrpic (Grenoble, France)

(xii) Service Central Hydrographique (Paris, France)

(xiii) Laboratoire National d'Hydraulique de Chatou (Chatou, S & O, France)

(xiv) Oseanograhisk Institutt, Universitet Blindern (Oslo, Norway)

(xv) Deutsches Hydrographisches Institut (Hamburg, Germany)

3. Preliminary Information for Planning:

For each beach under consideration for the landing, certain preliminary information is needed for planning; that is,

a. Charts, Maps and Tables:

Examine all available charts and maps of the area (translate and change scale to U. S. Standards. Make up master charts of the region and the landing area from best information available).

(i) Hydrographic charts, soundings, type of bottom, reefs, etc.

(ii) Topographic and geologic maps of shore and back country.

(iii) Pilot charts and coast pilot directions (Hydrographic Office). Prevailing winds and ocean currents in the geographic region. Notes on coast conditions.

(iv) Historical weather maps (Weather Bureau)

(v) Sea and Swell charts (Hydrographic Office), approximate wave conditions to be expected on the coastline selected at the season of the landing.

(vi) Predicted tides for interval of landing.

b. Supplementary Information:

Collect the following supplementary data from intelligence reports, magazines, travel books and diaries, research papers, geologic reports, tourist photos, recollections of travelers, etc.

(1) Prevailing Local Weather: Sea breeze, overcast, visibility, thundershowers, etc.

(2) Beach Conditions: Type and size of inter-tidal material (usually sand), trafficability, seasonal variations in width and slope, occurrence of bars, reefs, shoals, and kelp beds. Shorelines change in response to changing wave and surf conditions. Hydrographic surveys and geological examinations are usually made at random times and may portray the beach in a badly eroded condition or at the time of greatest seaward development. Bars, cusps, gravel deposits, and minor forms appear and disappear rapidly and frequently. On sand shores, recent accretions are usually soft but become harder under compaction by wave action. It is of the greatest importance that wave conditions be followed continuously after the aerial reconnaissance (Section VI B-4) to detect the possibility of a drastic change in the beach and that a final check on beach conditions be made just before the assault (Section VI B-6). This note on "beach intelligence" is included because much of the work on this subject in World War II apparently assumed beaches to be as static as the adjacent land forms. In reality they are about as variable as the surf which molds them.

(3) Surf Conditions: Intensity and variability in time and along beach, littoral currents, surf rips, effect of local winds.

(4) Tide Conditions: Range and time of tides.

(5) Ship Reports: Obtain all ship reports of sea and swell from the adjacent ocean areas which were used in preparing the Sea and Swell charts for the season of the landing and any other wave measurements made in the region.

c. Analysis of Available Data:

(1) Hindcast of Waves: From historical weather charts, hindcast waves to obtain the following information:

(i) Establish reliability of the wave forecasts by hindcasting the sea and swell at the time of past ship observations (Section VI B-3b).

(ii) If the hindcasts are shown to be reliable, select a typical year by inspection of historical weather maps and hindcast waves for each day to appraise seasonal variations.

(iii) Then, hindcast waves in the season of the landing in five different years, selected to represent the range of wave conditions.

(2) Local Weather: If supplementary information on local weather (Section VI B-3b) is conflicting or inconclusive, select areas of similar climate by inspection of weather charts, exposure, latitude, and so forth, and develop the probabilities as to fog, sea breeze, visibility, and other meteorological conditions.

(3) Forecast Surf Conditions:

(i) Construct wave refraction diagram using best information available on hydrography (Sections VI B-3a and VI B-3b).

(ii) Predict surf and currents produced by the range of conditions (length, period, direction) expected on the basis of Sea and Swell charts and wave hindcasts.

Analysis of this preliminary information should not assume that either the shoreline or the surf is static. Apparently conflicting statements may both be true but pertinent to different seasons or years. A sandy shoreline changes in slope and plan as the waves change, and waves are as variable daily, seasonally, and annually as other aspects of weather. A thorough study of all available records may only provide the probable beach and surf conditions at a certain season of the year, but probable conditions should be reduced to certainty by reconnaissance and forecasting immediately before and during the attack.

4. Preliminary Aerial Reconnaissance:

a. Oblique Photographs:

Single 45° oblique photos should be made to obtain data on surf, beach and beach exits, to check Preliminary Information gathered in Section VI B-3, selection of alternative landing areas, and provide basis for planning remainder of aerial reconnaissance.

(1) Photographic Specifications, preferably at 45° angle from one-half mile at sea. Sun angle for best photographic light. Time interval between photographs to give small overlap between adjacent exposures. Cameras: K-18, 24 inch and/or 20-inch (or newer cameras to replace these) for spotting critical areas such as harbors, bridges, suspected defenses.

(2) Flight Plan: Photographic weather, sun angle and the tide stage set a tight requirement on the time of the sortie. The desired combination of these factors will not occur frequently. Enemy anti-aircraft defenses will affect the altitude and distance of photography. Realistic flight plans must be developed and the plane crew and photographers must be thoroughly briefed on the requirements and objectives.

(3) Forecasting: Photographic weather, tide stages, and representative surf conditions.

b. Landfall Photographs:

Low altitude, long-range photograph is taken (from a submarine rather than an aeroplane, if possible) to cover coastline with small overlap. These photographs are necessary for briefing crews of ships and craft on the appearance of the shore. It has been found that boat crews have a very difficult time recognizing unfamiliar landfalls. This photography might be carried out later in the reconnaissance program, but a larger number of prints will be required, and the photography can be combined efficiently with the high-obliques. Forecasting and flight plans are the same as those in Section VI B-4a except that it is not necessary to consider surf conditions (or tide stage, except for exceptional cases of very great tidal range perhaps).

5. General Tactical Reconnaissance:

a. Aerial Reconnaissance:

(1) Timed Vertical Photographs on Planned Flight Lines Over Selected Beaches:

(a) Results Expected:

(i) Uncontrolled mosaic.

(ii) Beach and underwater gradients.

(iii) Inshore terrain.

(iv) Check on accuracy of forecasting of waves and weather.

(v) Location of underwater obstacles (to 10 feet approximately).

(vi) Information on littoral currents, surf rips, width of surf zone, variability of wave action.

(vii) Character of beach and trafficability.

(b) Flight and Photographic Plans: Eighty Percent Overlap, Minimum:

(i) Photographs at high tide, mid and low tide (minimum, 2 flights).

(ii) Time of day for best photography of waves.

(iii) Altitude 6,000 feet, if possible.

(iv) This will depend, upon tactical considerations. Photographs timed at 3 second intervals (approximately) to 0.1 seconds. K-17 (clock) 12-inch camera with clock reading to 0.1 second.

(c) Forecasting:

(i) Waves suitable for depth determination.

(ii) Photographic weather.

(iii) Tide.

The required combination of tide conditions, waves suitable for depth determination, photographic weather, and best time of day for wave photography will not occur frequently. Reconnaissance flights should not be sent out when conditions are unfavorable; favorable conditions should not be missed. Tight coordination of aerologists, photographers, photo interpreters, and flight crews is essential and all key personnel must understand the requirements to be met.

(2) Timed Vertical Photography: To roughly check accuracy of wave forecasting. Same requirements as in Item (1) in this section but limited to areas where waves are easily analyzed. The results from this intelligence are of a rough nature and serve mainly to prevent blunders in forecasting due to unforeseen conditions. If the beach profile has been obtained by a U.D.T. then this method will produce more accurate results.

Note: The experience with southerly swell at Camp Pendleton serves to emphasize the necessity for wave measurements and the dangers of depending on forecasting from weather maps alone. The swell from the Southern Hemisphere, arriving in Southern California cannot be forecast at present because wind data are not available for the generating area. Forecasts based on winds in the areas covered by synoptic charts have been erroneous as to the period, height, and direction of the waves and the direction of littoral current.

(a) Results Expected:

(i) Approximate breaker heights,

(ii) Wave periods and comparison of both of these with wave forecasts.

(b) Flight and Photographic Plans:

(i) Any stage of tide

(ii) Maximum overlap (minimum of 80%) to follow individual waves as far as possible.

(iii) Scale-same as VI B-5a(a).

(iv) Time of day for best lighting of water surface.

(v) Five passes over selected area on each sortie.

(vi) Five sorties, including two tested under Item (1) in this section.

(c) Forecasting:

(i) Waves. The most effective check on the accuracy of wave forecasting will be obtained from flights before and after a predicted change in surf.

(ii) Photographic weather.

(3) Obliques: Single photos of specific areas for detail. Here, the peek-a-boo camera is at its best. For example, the F-56, 20-inch camera may be used for detailing the subject; the F-56, 8-1/4-inch shows its relation to the surroundings.

(4) Low altitude steros for picking up scarps, etc.

b. Ground Reconnaissance:

After the detailed photos have been examined and interpreted, U.D.T. swimmer and rubber-boat reconnaissance units should be sent onto the beach to check the information. They should perform the hydrographic surveying and spend as much time as possible on checking the condition of exit roads, defenses, determining beach trafficability and looking for mines and traps. Take sand samples at selected points, check fresh water supply. After the photos are examined, there will be doubt as to the meaning of certain specks, or as to whether amphibians can pass rocks off the beach at some tide stages, etc. The investigation of specific items such as these are U.D.T. business.

(1) Planning: Use landfalls and obliques for familiarization (Section VI B-4). Obtain underwater gradient and submerged obstacles from section VI B-5. Tide conditions and lighting (moon or sun). Prepare detailed topography and hydrography on basis of all available data.

(2) Forecasting:

(i) Wave and surf conditions

(ii) Local winds

(iii) Littoral currents and surf rips.

(3) Observations:

(i) Survey profile along representative ranges.

(ii) Obtain samples of beach sand at points selected from examination of photographs obtained in Section VI B-5a.

(iii) Measure beach traffic capacity at selected points.

(iv) Measure breaker height, period, and distance from shore.

(v) Measure direction and strength of littoral current.

(vi) Observe details of bottom in, and near, the surf zone; that is, bars, pot-holes, coral heads, rocks, gravel, and other features. Depths over bars, shoals and obstacles.

(vii) Observe details of shoreface (cusps, soft surface sand, etc.) of berm, and backshore.

(4) Installation of Portable Wave Recorder: Wave recorders for shallow waters have been developed and techniques for installing by U.D.T.'s partially worked out. An instrument of this type could be installed under cover of darkness and records obtained. A telemetering recorder has been developed which could be installed to give a continuous record which could be recorded aboard a ship or small craft.

c. Hydrographic and Topographic Charts:

(i) Prepare a mosaic of landing area and correct hydrographic and topographic charts to include all data obtained up to this point.

(ii) Prepare refraction diagrams on basis of latest hydrographic information.

6. Final Tactical Reconnaissance:

a. Vertical Photographs:

Obtain timed vertical photographs as in Section VI B-5a, at latest possible moment before H-hour, but in time to be useful in planning the assault, to check the accuracy of the surf forecast. Photographs taken from land by reconnaissance teams, if feasible.

b. Oblique Photographs:

These photographs should be taken as in Section VI B-a to spot changes in general beach conditions or last-minute defense measures.

7. Wave Forecasting:

Forecast the height, period and type of the breakers, the width and description of the surf zone and the direction and strength of the littoral currents for reconnaissance and for landing conditions on first assault.

a. From Prognostic Weather Charts:

Tentative wave forecasts for a few days ahead of the more reliable forecasts made from Synoptic Charts.

b. From Synoptical Weather Charts:

Requires weather maps covering the areas which may contribute waves to the shoreline selected. Weather ships or planes should obtain pressure data for areas not otherwise reported. Maps should be drawn at 6-hour intervals for several days prior to operational wave forecast.

c. From Nearby Wave Measurements:

Wave measurements made in adjacent areas may be projected both in time and distance to provide wave data in the locality under study (see Section VII).

Submarines or surface ships, at sea or, possibly, adjacent points on land, can operate wave measuring equipment, and the measurements may be projected to the selected beach and, on the basis of the previous sequence of wave conditions, the forecast in time may be made. (Note: On some shorelines, local wind waves prevail and the wave conditions are as variable as the winds. However, on many shorelines the breakers are caused by waves generated many miles at sea and there is a gradual transition from one condition to another.

The intelligence obtained in Section VI B-5 should provide a basis for forecasting from wave measurements.

d. Using the Wave Forecasts, an estimation of changes, if any, in the beach should be made.

8. Wave Measurements:

a. Prior to Landing:

(1) Aerial Method (Section VI B-5a). Not very accurate, but a check.

(2) Reconnaissance Parties (Section VI B-5a).

(3) At Sea or Adjacent Friendly Shores: Instruments have been developed (but are not standard equipment) for wave measurements at sea in deep water. Several recorders are available for wave measurements in coastal areas or shallow seas. Measurements 50 to 100 miles at sea will give from 1 to 3 hours advance information on general wave and surf conditions.

b. During Assault Phase: Wave recorders can be provided either to (i) be dropped in a fixed position and report by radio (developed but not standard), or (ii) be towed by command ships. When surf conditions are difficult, wave measurements will give warning of group of high waves and of low waves.

c. Troop Landings and Cargo Phase: After the initial landing, a wave recorder with pressure pick-up seaward of the breakers and recorders on shore should be used to control ship-to-shore traffic and return.

9. Tidal Data:

a. Predictions of Tides:

Predictions of tidal variations are available for almost all shorelines, and in general, these forecasts are reliable. However, there are some shorelines for which tidal data are not available or for which the predictions are subject to large errors because of meteorological conditions.

b. Direct Measurements:

Self-contained tide gages have been developed (University of California) which will record for two days. These instruments can be placed and recovered by submarine.

c. Aerial Photographs:

Approximate determinations of tidal elevations can be made from aerial photographs, but this method would be suitable only to determine gross errors in tidal predictions or large wind effects.

10. Departmental Task Breakdown:

a. Aerologists:

Weather forecasts, synoptic weather charts, and prognostic weather charts must be kept up to date and available to wave forecasters, pilots and photographers.

(i) Particular consideration should be given to aerial photographic weather rather than just flying weather, especially for high altitude photography.

(ii) Conversely, fog conditions may be of help to U.D.T. operations and should be forecast.

b. Wave Forecasters:

(i) These forecasters must keep their forecasts up to date as to both period and direction so that (with refraction diagrams) the maximum surf can be predicted and used to advantage in the depth-determination photography. (Where the depth determination photos are wanted for refraction diagram contours, refraction approximations can be used for predictions.)

(ii) The U.D. Teams must be briefed as to the surf they will encounter.

(iii) Landing conditions can usually be forecast at least two days ahead for most locations.

c. Engineering Officer:

(i) Calculation of tides--tide stage plotted as a curve for any time of day or night for quick reading.

(ii) Preparation of refraction diagrams for probable periods and directions.

(iii) Bottom contour maps from photographic interpretation and UDT information. (The two must be reconciled.)

(iv) Transferring of information from the aerial mosaics to charts for specific uses.

(v) Operation and interpretation of wave and tide recorders.

(vi) Evaluation of docking facilities.

d. Photo Interpreters:

(1) Photo Analysis:

(i) Slope of beach (above and below water), and number of offshore bars or reefs.

(ii) Composition and trafficability of beach.

(iii) Kind of vegetation at back beach.

(iv) Beach exits.

(v) Beach defenses and underwater obstacles.

(vi) Approximate size of surf for comparison with forecasts. This can best be done when beach profiles have been obtained by U.D.T.'s.

(vii) Size and location of scarps, cusps, surf rips, etc.

(2) Underwater Contours: The plotting of the underwater contours must be completed, using the underwater depth-determination photos (K-17, or similar camera).

(3) Miscellaneous: Constant lookout for anything unusual that may be a hazard (rocks exposed through sand, traps, defenses).

e. Photographers and Photo Technicians:

The best possible photo work must be done at all times as the proper conditions may never reoccur. This means constant vigil on the many details connected with aerial photography. Items which cause trouble and must be checked constantly are:

(i) Proper combination of exposure, shutter speed, film speed, filters, etc.

(ii) Adequate vacuum (read on line meter). (Especially important on the larger cameras.)

(iii) Camera clocks--winding, light source, exposure (in printing, clock can be dodged for correct exposure).

(iv) Take up warning--showing whether film is moving ahead properly and when all film is used up.

(v) Intervalometer. Since most intervalometers can only be set for even seconds, on lower altitude photos

(and on depth-determination photos) more photos can be secured by holding the switch down so that the camera fires as soon as it has recycled.

(vi) Viewfinder and contact between photographer and pilot, so that the right picture coverage is secured in the fewest number of runs.

(vii) Photolab techniques. Tendencies to use old developer and/or developer at the wrong temperature must be curbed. A "grainy" negative may be useless to the photo interpreters.

f. Photoplane Operations Officer:

The success of the reconnaissance depends on the judgment of this man and on the cooperation and advice that he gets from the other members of the organization. Although he may have definite instructions as to the type and scale of photo wanted, he will have to decide on details of flight and photographic plan with the pilot and photographic officer.

(i) Kind and focal length of camera.

(ii) Kind of plane--if oblique photos, whether there are photo ports in each side and the movement of engine exhaust gages with respect to these ports (may make photo fuzzy).

(iii) Enemy defenses and necessary altitude of plane for desired scale of photos.

(iv) Direction of wind (may be desirable to fly into the wind for more photo coverage or with it to pass defenses quickly).

(v) Nature of cloud cover, time for haze to burn off, density of cloud cover(number of runs necessary to get photos through holes).

(vi) Direction of sun--especially for oblique photos.

(vii) Tide stage when plane arrives over target.

g. Underwater Demolition Teams:

(1) Reconnaissance Sortie: A reconnaissance sortie should be made some time in advance of the operation (around D minus 10). At this time the following information can be secured:

(i) Profile of beach to-30 feet below datum.

(ii) Condition of bottom and bars.

(iii) Sand samples. Trafficability notes.

(iv) Beach defenses.

(v) Beach exits.

(vi) Ground cover (underbrush).

(vii) Check list of items furnished by PIC.

(2) Check Sortie: Immediately before the operation (H minus 12) it probably will be necessary for the U. D. Teams to make a last minute sortie. At this time, their work should include checks on:

(i) Changes in beach profile (i.e., size and location of bars).

(ii) Surf size (comparison with forecast) height of breaker.

(iii) Trafficability of beach (changes with cutting or filling actions, scarps, etc.).

(iv) Check on exit blocks, defenses.

(v) Check any items requested by PIC for identification or confirmation.

(vi) Find and destroy underwater obstacles to landing ships and craft.

(vii) Location and size of surf rips.

(viii) Littoral currents.

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~~RESTRICTED Security Information~~

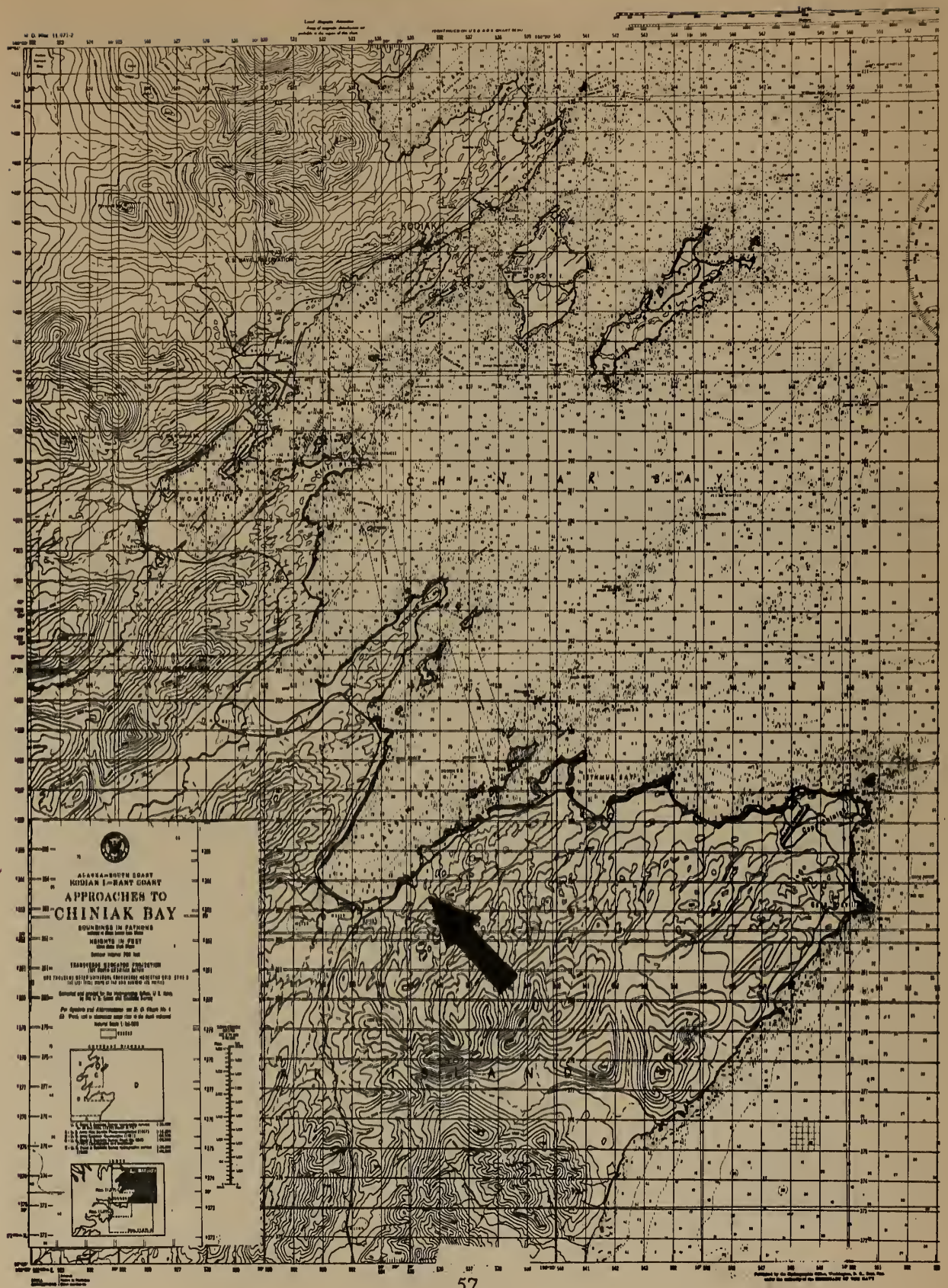





FIGURE VI B-1

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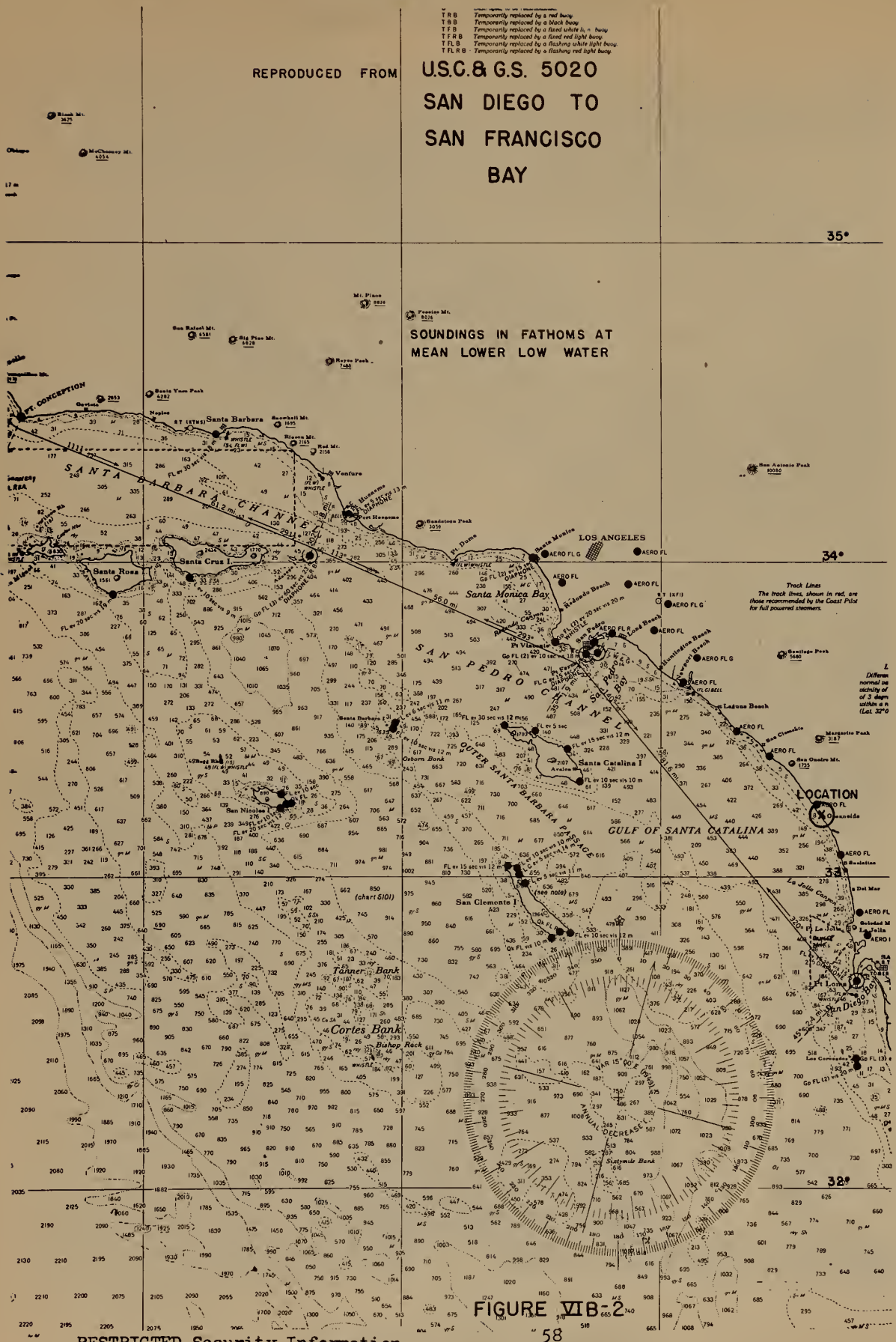

REPRODUCED FROM U.S.C.& G.S. 5020
SAN DIEGO TO SAN FRANCISCO BAY
SOUNDINGS IN FATHOMS AT MEAN LOWER LOW WATER
TRB Temporarily replaced by a red buoy.
TBB Temporarily replaced by a black buoy.
TFB Temporarily replaced by a fixed white lt. = buoy
TFRB Temporarily replaced by a fixed red light buoy.
TFLB Temporarily replaced by a flashing white light buoy.
TFLRB Temporarily replaced by a flashing red light buoy.
35°
34°
33°
32°
PT. CONCEPTION
Santa Barbara
Ventura
Santa Rosa
Santa Cruz I.
SANTA BARBARA CHANNEL
Santa Monica Bay
LOS ANGELES
Redondo Beach
San Pedro
Long Beach
Laguna Beach
SAN PEDRO CHANNEL
Santa Catalina I.
Avalon
Osborn Bank
San Nicolas
OUTER SANTA BARBARA PASSAGE
GULF OF SANTA CATALINA
San Clemente I.
Tanner Bank
Cortes Bank
Bishop Rock
Sixtymile Bank
LOCATION
San Diego
Track Lines
The track lines, shown in red, are those recommended by the Coast Pilot for full powered steamers.
56.0 mi
61.2 mi
(chart 5101)
(see note)

FIGURE VIB-2

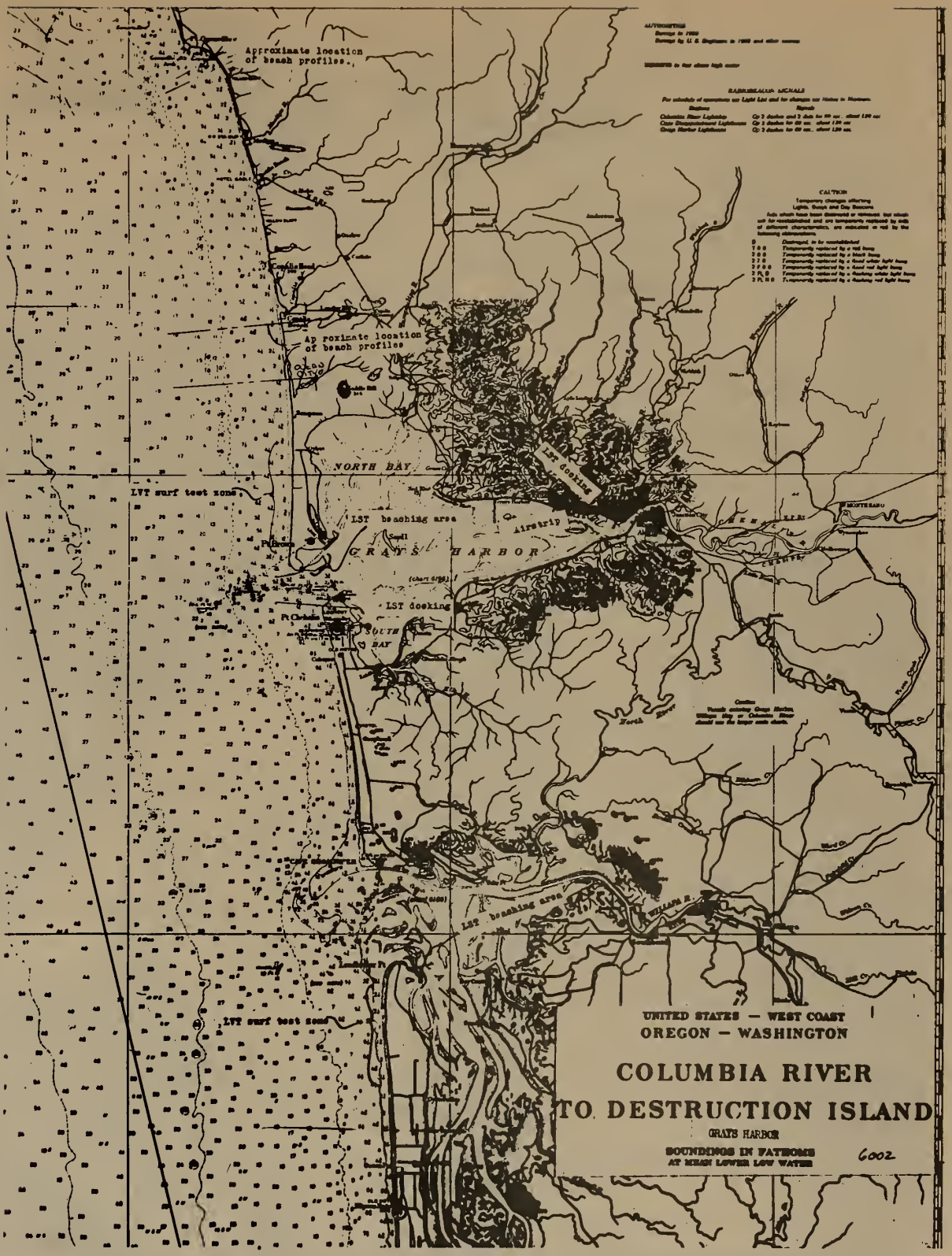

Approximate location of beach profiles.
Approximate location of beach profiles
NORTH BAY
LST docking
LVT surf test zone
LST beaching area
Airstrip
GRAYS HARBOR
LST docking
SOUTH BAY
LST beaching area
LVT surf test zone
UNITED STATES — WEST COAST
OREGON — WASHINGTON
COLUMBIA RIVER
TO DESTRUCTION ISLAND
GRAYS HARBOR
SOUNDINGS IN FATHOMS
AT MEAN LOWER LOW WATER
6002

FIGURE VI B-3

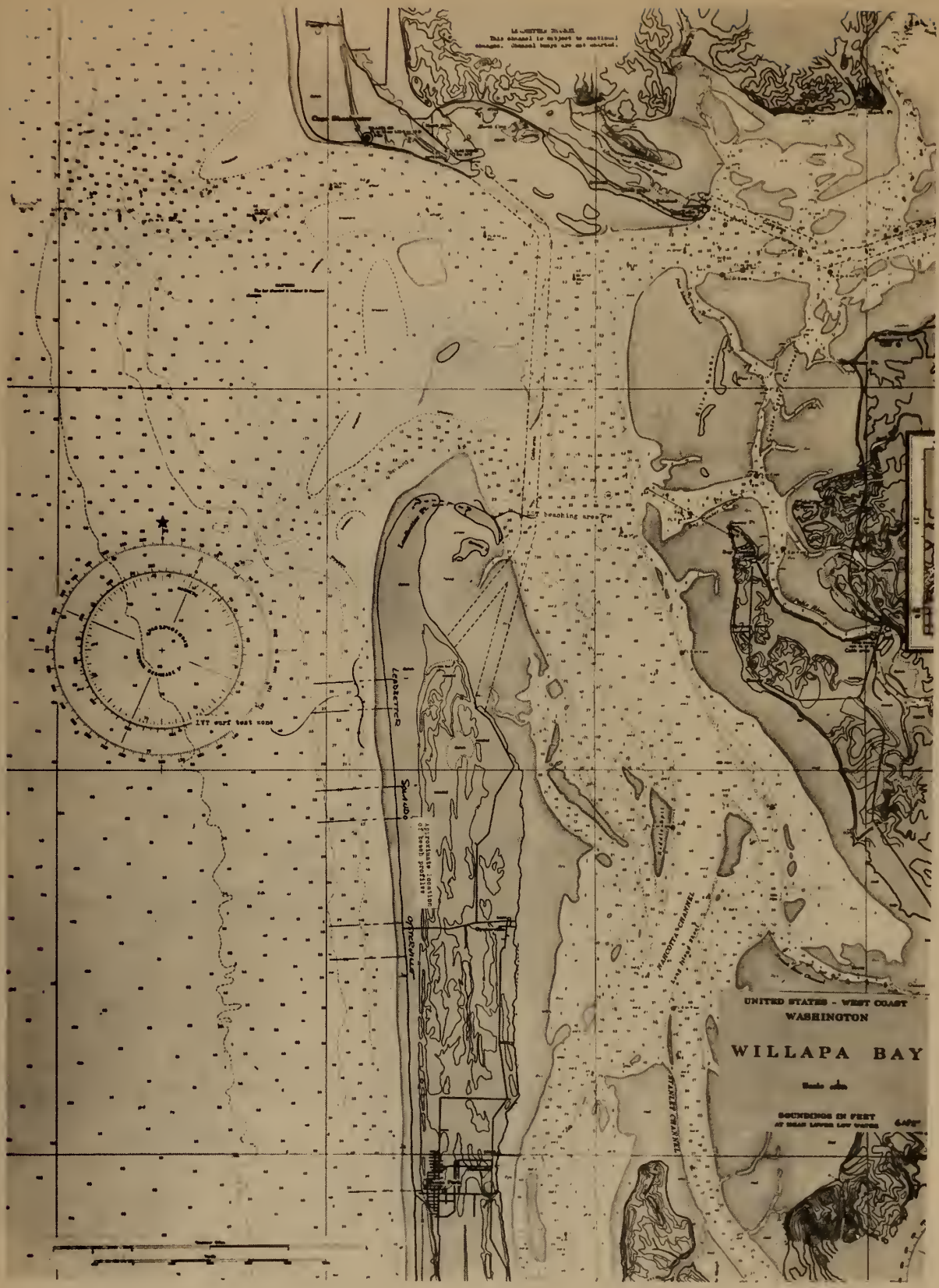

UNITED STATES - WEST COAST
WASHINGTON
WILLAPA BAY
SOUNDINGS IN FEET
Leadbetter
Shoalwater
Peacock Spit
Long Island
Stackpole
Channel
Middle Channel
Jetty
Breaking area
177 surf test zone

FIGURE VIB-4

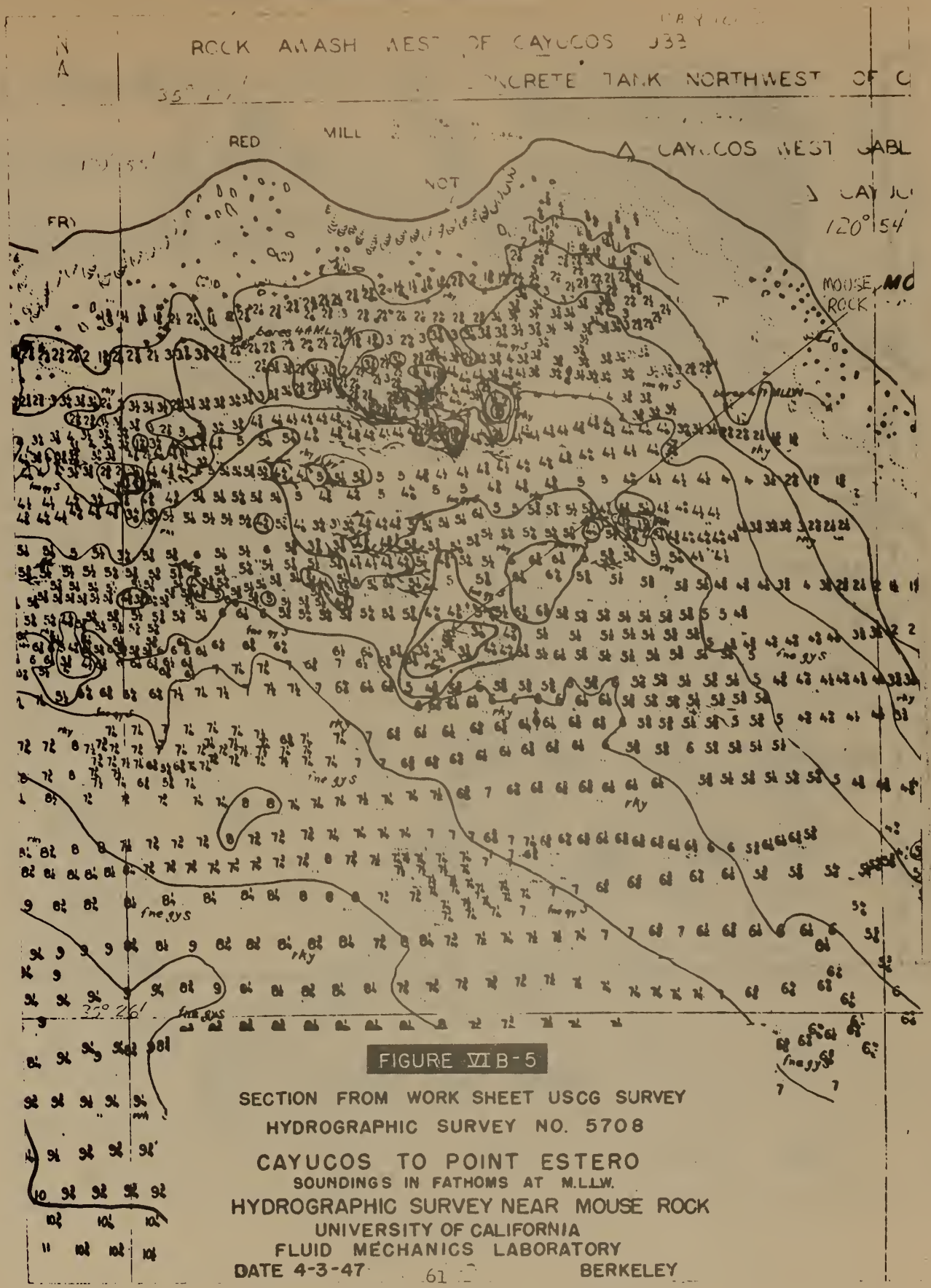



FIGURE VI B-5

SECTION FROM WORK SHEET USCG SURVEY
HYDROGRAPHIC SURVEY NO. 5708

CAYUCOS TO POINT ESTERO
SOUNDINGS IN FATHOMS AT M.L.L.W.
HYDROGRAPHIC SURVEY NEAR MOUSE ROCK
UNIVERSITY OF CALIFORNIA
FLUID MECHANICS LABORATORY
DATE 4-3-47 BERKELEY

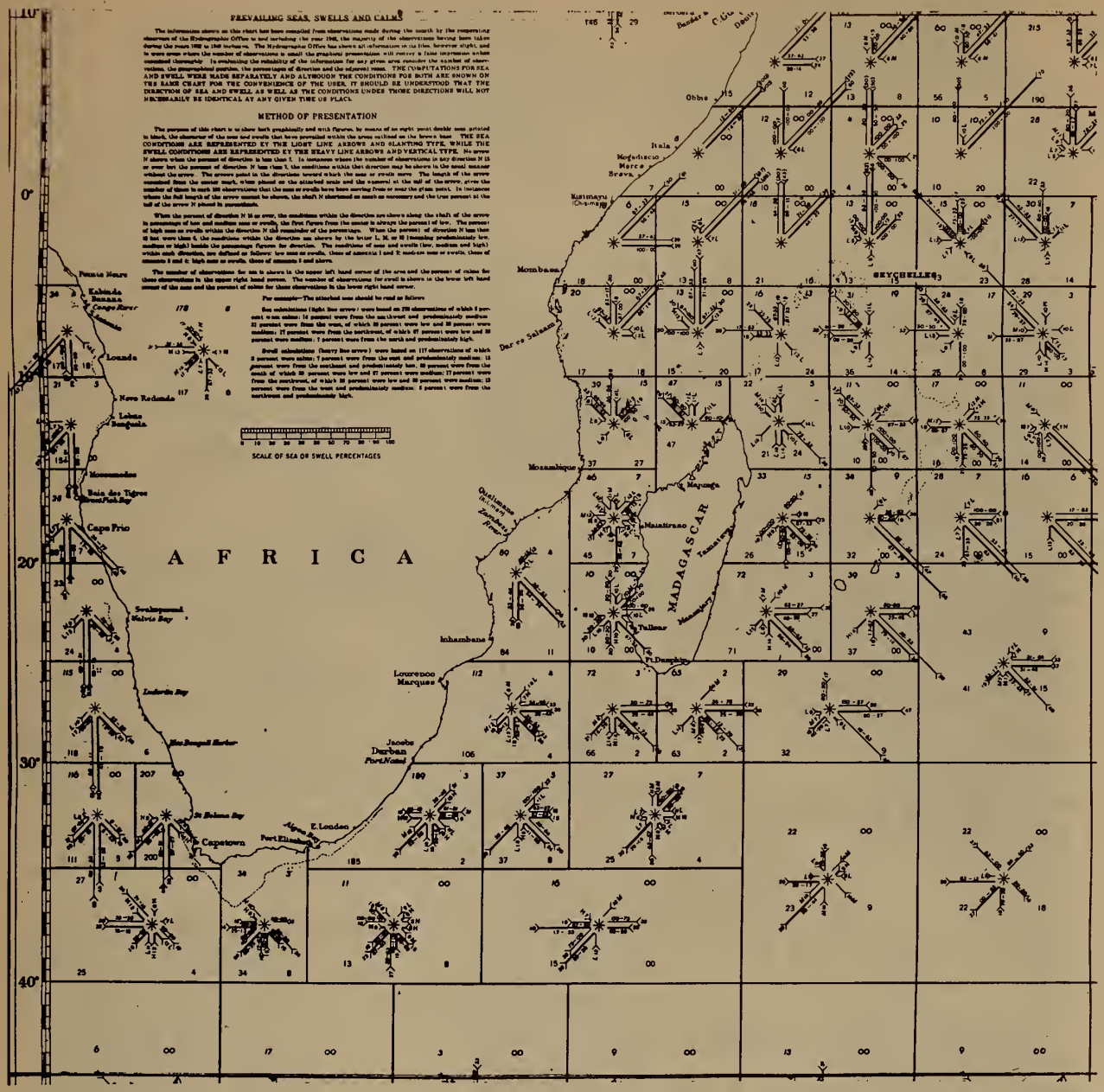

PREVAILING SEAS, SWELLS AND CALMS
METHOD OF PRESENTATION
SCALE OF SEA OR SWELL PERCENTAGES
A F R I C A
MADAGASCAR
Pointe Noire
Kabinda
Banana
Congo River
Luanda
Novo Redondo
Lobito
Benguela
Mossamedes
Baia dos Tigres
Cape Frio
Swakopmund
Walvis Bay
Luderitz Bay
Alexander Bay
St Helena Bay
Cape town
Port Elizabeth
E London
Durban
Port Natal
Lourenco Marques
Inhambane
Beira
Mozambique
Mayunga
Maintirano
Tulear
Ft Dauphin
Diego Suarez
Mombasa
Obbia
Mogadiscio
Merca
Brava
Kismayu
Ruia
SEYCHELLES
Aldabra
Farquhar
Desroches
Amirantes
Providence
Cosmoledo
Assumption
Astove
Aldabra
Farquhar
Desroches
Amirantes
Providence
Cosmoledo
Assumption
Astove

FIGURE VIB-6

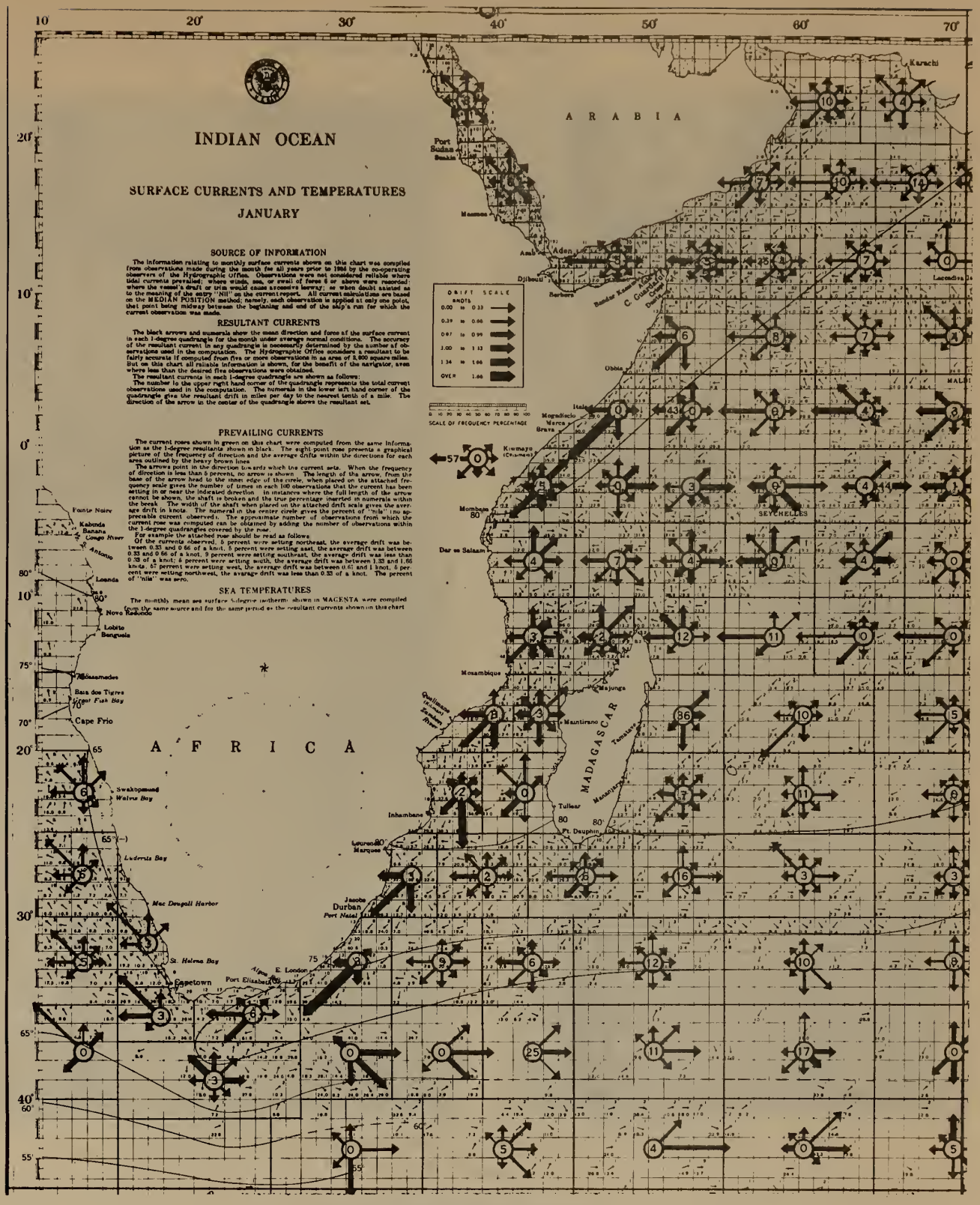

INDIAN OCEAN
SURFACE CURRENTS AND TEMPERATURES
JANUARY
SOURCE OF INFORMATION
The information relating to monthly surface currents shown on this chart was compiled from observations made during the month for all years prior to 1956 by the co-operating observers of the Hydrographic Office. Observations were not considered reliable where tidal currents prevailed; where winds, sea, or swell of force 6 or above were recorded; where the vessel's draft or trim would cause excessive leeway; or when doubt existed as to the meaning of the entry "Nil" on the current report. All current calculations are based on the MEDIAN POSITION method; namely, each observation is applied at only one point, that point being midway between the beginning and end of the ship's run for which the current observation was made.
RESULTANT CURRENTS
The black arrows and numerals show the mean direction and force of the surface current in each 1-degree quadrangle for the month under average normal conditions. The accuracy of the resultant current in any quadrangle is necessarily determined by the number of observations used in the computation. The Hydrographic Office considers a resultant to be fairly accurate if computed from five or more observations in an area of 3,600 square miles. But on this chart all reliable information is shown, for the benefit of the navigator, even where less than the desired five observations were obtained.
The resultant currents in each 1-degree quadrangle are shown as follows:
The number in the upper right hand corner of the quadrangle represents the total current observations used in the computation. The numerals in the lower left hand corner of the quadrangle give the resultant drift in miles per day to the nearest tenth of a mile. The direction of the arrow in the center of the quadrangle shows the resultant set.
PREVAILING CURRENTS
The current roses shown in green on this chart were computed from the same information as the 1-degree resultants shown in black. The eight-point rose presents a graphical picture of the frequency of direction and the average drifts within the directions for each area outlined by the heavy brown lines.
The arrows point in the direction towards which the current sets. When the frequency of direction is less than 5 percent, no arrow is shown. The length of the arrow, from the base of the arrow head to the inner edge of the circle, when placed on the attached frequency scale gives the number of times in each 100 observations that the current has been setting in or near the indicated direction. In instances where the full length of the arrow cannot be shown, the shaft is broken and the true percentage inserted in numerals within the break. The width of the shaft when placed on the attached drift scale gives the average drift in knots. The numeral in the center circle gives the percent of "nils" (no appreciable current) observed. The approximate number of observations from which the current rose was computed can be obtained by adding the number of observations within the 1-degree quadrangles covered by the rose.
For example the attached rose should be read as follows:
Of the currents observed, 5 percent were setting northeast, the average drift was between 0.33 and 0.66 of a knot, 5 percent were setting east, the average drift was between 0.33 and 0.66 of a knot, 9 percent were setting southeast, the average drift was less than 0.33 of a knot, 6 percent were setting south, the average drift was between 1.33 and 1.66 knots, 67 percent were setting west, the average drift was between 0.67 and 1 knot, 6 percent were setting northwest, the average drift was less than 0.33 of a knot. The percent of "nils" was zero.
SEA TEMPERATURES
The monthly mean sea surface Celsius isotherms shown in MAGENTA were compiled from the same source and for the same period as the resultant currents shown on this chart
DRIFT SCALE
KNOTS
0.00 to 0.33
0.34 to 0.66
0.67 to 0.99
1.00 to 1.33
1.34 to 1.66
OVER 1.66
SCALE OF FREQUENCY PERCENTAGE
ARABIA
AFRICA
MADAGASCAR
SEYCHELLES
Karachi
Aden
Djibouti
Berbera
Port Sudan
Suakin
Massawa
Obbia
Mogadiscio
Merca
Brava
Kismayo
Mombasa
Dar es Salaam
Mozambique
Majunga
Mantirano
Tamatave
Tulliar
Ft. Dauphin
Inhambane
Lourenco Marques
Durban
Port Natal
Port Elizabeth
East London
Capetown
St. Helena Bay
Mac Dougall Harbor
Walvis Bay
Swakopmund
Lüderitz Bay
Cape Frio
Baia dos Tigres
Tiger Fish Bay
Mossamedes
Novo Redondo
Lobito
Benguela
Loanda
Banana
Congo River
Kabinda
Pointe Noire

FIGURE VI B-7

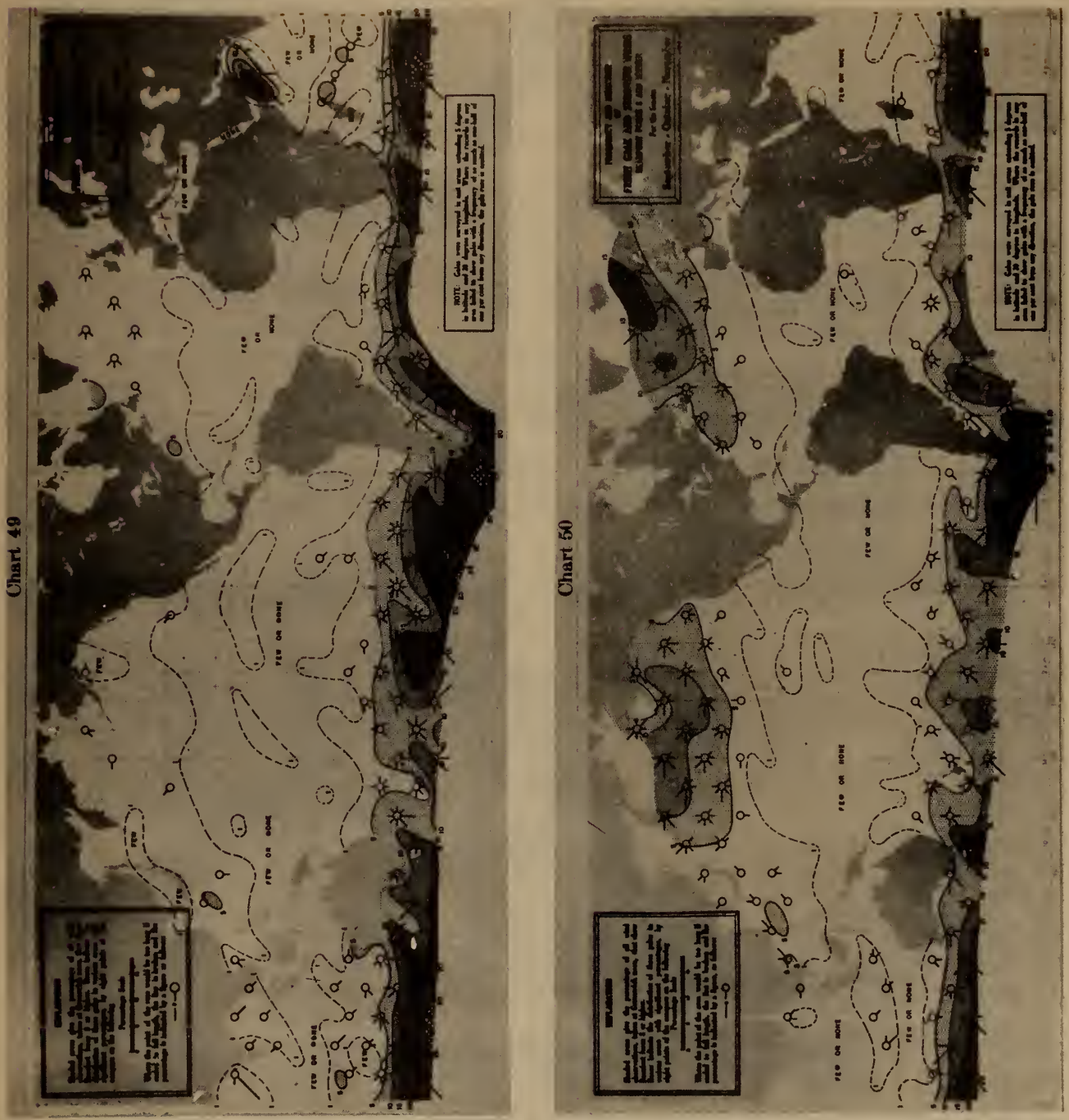





FIGURE VI B-8

| APRIL | | | | | MAY | | | | | JUNE | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Day | High | | Low | | Day | High | | Low | | Day | High | | Low | |
| | Time | Ht. | Time | Ht. | | Time | Ht. | Time | Ht. | | Time | Ht. | Time | Ht. |
| | h. m. | ft. | h. m. | ft. | | h. m. | ft. | h. m. | ft. | | h. m. | ft. | h. m. | ft. |
| 1 Su | 7 55<br>17 00 | 6.7<br>8.1 | 0 25<br>11 28 | 1.1<br>5.9 | 1 Tu | 7 53<br>19 17 | 8.1<br>7.6 | 0 56<br>13 20 | 1.4<br>4.8 | 1 F | 8 14<br>21 13 | 9.7<br>7.7 | 1 42<br>15 13 | 3.4<br>2.5 |
| 2 M | 8 46<br>19 26 | 7.4<br>8.0 | 1 44<br>13 21 | 0.9<br>5.4 | 2 W | 8 32<br>20 32 | 8.7<br>8.0 | 1 55<br>14 36 | 1.7<br>3.7 | 2 Sa | 8 51<br>22 04 | 10.1<br>7.9 | 2 27<br>16 07 | 3.9<br>1.7 |
| 3 Tu | 9 20<br>20 45 | 8.2<br>8.5 | 2 46<br>14 49 | 0.7<br>4.4 | 3 Th | 9 06<br>21 26 | 9.3<br>8.4 | 2 43<br>15 33 | 2.0<br>2.7 | 3 Su | 9 25<br>22 52 | 10.5<br>8.0 | 3 09<br>16 54 | 4.4<br>1.2 |
| 4 W | 9 51<br>21 40 | 8.8<br>9.0 | 3 35<br>15 51 | 0.6<br>3.4 | 4 F | 9 36<br>22 14 | 9.9<br>8.6 | 3 27<br>16 20 | 2.4<br>1.9 | 4 M | 9 59<br>23 38 | 10.6<br>8.1 | 3 48<br>17 38 | 4.9<br>1.0 |
| 5 Th | 10 20<br>22 26 | 9.4<br>9.2 | 4 16<br>16 38 | 0.8<br>2.5 | 5 Sa | 10 06<br>23 00 | 10.3<br>8.6 | 4 04<br>17 05 | 2.9<br>1.3 | 5 Tu | 10 30<br>. .. | 10.7<br>. . | 4 26<br>18 20 | 5.4<br>0.7 |
| 6 F | 10 49<br>23 09 | 9.8<br>9.2 | 4 53<br>17 21 | 1.2<br>1.7 | 6 Su | 10 35<br>23 42 | 10.5<br>8.5 | 4 38<br>17 49 | 3.4<br>0.9 | 6 W | 0 22<br>11 00 | 8.1<br>10.7 | 5 04<br>19 00 | 5.7<br>0.7 |
| 7 Sa | 11 17<br>23 51 | 10.1<br>9.0 | 5 26<br>18 02 | 1.7<br>1.3 | 7 M | 11 03<br>. .. | 10.6<br>. . | 5 09<br>18 30 | 4.0<br>0.8 | 7 Th | 1 06<br>11 32 | 8.1<br>10.7 | 5 43<br>19 34 | 6.0<br>0.7 |
| 8 Su | 11 45<br>. .. | 10.3<br>. . | 5 56<br>18 44 | 2.3<br>1.0 | 8 Tu | 0 24<br>11 31 | 8.3<br>10.7 | 5 40<br>19 08 | 4.5<br>0.7 | 8 F | 1 45<br>12 05 | 8.1<br>10.6 | 6 23<br>20 06 | 6.1<br>0.8 |
| 9 M | 0 31<br>12 11 | 8.7<br>10.4 | 6 24<br>19 22 | 2.9<br>1.0 | 9 W | 1 08<br>12 00 | 8.0<br>10.6 | 6 10<br>19 46 | 4.9<br>0.8 | 9 Sa | 2 23<br>12 38 | 8.1<br>10.5 | 7 05<br>20 35 | 6.1<br>1.0 |
| 10 Tu | 1 11<br>12 37 | 8.3<br>10.4 | 6 49<br>20 00 | 3.5<br>1.0 | 10 Th | 1 50<br>12 29 | 7.8<br>10.5 | 6 42<br>20 23 | 5.3<br>1.0 | 10 Su | 2 56<br>13 14 | 8.1<br>10.2 | 7 49<br>21 04 | 6.0<br>1.2 |
| 11 W | 1 51<br>13 03 | 7.8<br>10.3 | 7 14<br>20 38 | 4.0<br>1.2 | 11 F | 2 35<br>12 59 | 7.5<br>10.3 | 7 15<br>21 00 | 5.5<br>1.2 | 11 M | 3 31<br>13 54 | 8.2<br>9.8 | 8 38<br>21 35 | 5.8<br>1.5 |
| 12 Th | 2 37<br>13 29 | 7.2<br>10.1 | 7 41<br>21 19 | 4.5<br>1.5 | 12 Sa | 3 22<br>13 29 | 7.3<br>10.0 | 7 54<br>21 37 | 5.8<br>1.5 | 12 Tu | 4 06<br>14 39 | 8.3<br>9.1 | 9 32<br>22 12 | 5.5<br>2.0 |
| 13 F | 3 29<br>13 55 | 6.7<br>9.7 | 8 11<br>22 09 | 5.1<br>1.9 | 13 Su | 4 20<br>14 05 | 7.2<br>9.4 | 8 45<br>22 20 | 6.0<br>1.8 | 13 W | 4 48<br>15 37 | 8.5<br>8.2 | 10 37<br>22 54 | 5.2<br>2.6 |
| 14 Sa | 4 56<br>14 27 | 6.3<br>9.2 | 8 51<br>23 08 | 5.6<br>2.1 | 14 M | 5 30<br>14 50 | 7.2<br>8.7 | 9 52<br>23 07 | 6.0<br>2.1 | 14 Th | 5 39<br>17 05 | 8.8<br>7.4 | 11 48<br>23 41 | 4.6<br>3.2 |
| 15 Su | 6 56<br>15 05 | 6.5<br>8.5 | 9 59<br>. .. | 6.1<br>. . | 15 Tu | 6 31<br>15 54 | 7.5<br>8.0 | 11 07<br>. .. | 5.9<br>. . | 15 F | 6 28<br>19 08 | 9.2<br>7.0 | . ..<br>13 05 | . .<br>3.7 |
| 16 M | 8 01<br>16 15 | 7.0<br>7.8 | 0 14<br>11 29 | 2.3<br>6.2 | 16 W | 7 17<br>17 46 | 8.0<br>7.4 | 0 00<br>12 26 | 2.4<br>5.2 | 16 Sa | 7 16<br>20 39 | 9.8<br>7.2 | 0 31<br>14 19 | 3.8<br>2.7 |
| 17 Tu | 8 36<br>18 36 | 7.6<br>7.5 | 1 15<br>12 57 | 2.2<br>5.8 | 17 Th | 7 53<br>19 38 | 8.6<br>7.5 | 0 50<br>13 38 | 2.7<br>4.3 | 17 Su | 8 04<br>21 47 | 10.4<br>7.6 | 1 24<br>15 29 | 4.4<br>1.7 |
| 18 W | 9 04<br>20 05 | 8.2<br>7.9 | 2 08<br>14 11 | 2.0<br>5.0 | 18 F | 8 25<br>20 50 | 9.3<br>7.8 | 1 37<br>14 43 | 3.0<br>3.2 | 18 M | 8 51<br>22 46 | 10.9<br>8.0 | 2 18<br>16 29 | 4.9<br>0.7 |
| 19 Th | 9 28<br>21 07 | 8.7<br>8.4 | 2 48<br>15 09 | 2.0<br>4.0 | 19 Sa | 8 57<br>21 49 | 9.9<br>8.1 | 2 22<br>15 39 | 3.3<br>2.2 | 19 Tu | 9 37<br>23 40 | 11.2<br>8.3 | 3 13<br>17 24 | 5.4<br>0.0 |
| 20 F | 9 52<br>21 58 | 9.3<br>8.7 | 3 26<br>15 58 | 2.2<br>3.0 | 20 Su | 9 30<br>22 41 | 10.5<br>8.3 | 3 04<br>16 32 | 3.7<br>1.2 | 20 W | 10 23<br>. .. | 11.5<br>. . | 4 13<br>18 16 | 5.7<br>-0.5 |
| 21 Sa | 10 17<br>22 45 | 9.8<br>8.9 | 4 00<br>16 44 | 2.4<br>2.0 | 21 M | 10 04<br>23 34 | 11.0<br>8.3 | 3 48<br>17 26 | 4.2<br>0.4 | 21 Th | 0 30<br>11 11 | 8.5<br>11.5 | 5 19<br>19 04 | 5.9<br>-0.8 |
| 22 Su | 10 44<br>23 32 | 10.3<br>8.8 | 4 35<br>17 31 | 2.8<br>1.2 | 22 Tu | 10 42<br>. .. | 11.3<br>. . | 4 32<br>18 19 | 4.7<br>-0.1 | 22 F | 1 17<br>11 58 | 8.7<br>11.2 | 6 22<br>19 49 | 5.8<br>-0.8 |
| 23 M | 11 14<br>. .. | 10.7<br>. . | 5 10<br>18 19 | 3.3<br>0.5 | 23 W | 0 26<br>11 21 | 8.3<br>11.4 | 5 20<br>19 09 | 5.1<br>-0.5 | 23 Sa | 2 00<br>12 45 | 8.9<br>10.9 | 7 22<br>20 30 | 5.5<br>-0.4 |
| 24 Tu | 0 20<br>11 46 | 8.5<br>11.0 | 5 46<br>19 08 | 3.8<br>0.1 | 24 Th | 1 18<br>12 02 | 8.2<br>11.2 | 6 13<br>19 59 | 5.4<br>-0.6 | 24 Su | 2 40<br>13 32 | 9.0<br>10.3 | 8 17<br>21 09 | 5.1<br>0.2 |
| 25 W | 1 10<br>12 20 | 8.2<br>11.1 | 6 23<br>19 59 | 4.2<br>-0.1 | 25 F | 2 10<br>12 45 | 8.2<br>10.9 | 7 09<br>20 46 | 5.6<br>-0.4 | 25 M | 3 20<br>14 20 | 9.0<br>9.5 | 9 08<br>21 47 | 4.8<br>1.0 |
| 26 Th | 2 01<br>12 57 | 7.8<br>10.9 | 7 04<br>20 52 | 4.7<br>0.0 | 26 Sa | 3 02<br>13 30 | 8.1<br>10.3 | 8 08<br>21 33 | 5.6<br>0.0 | 26 Tu | 4 00<br>15 16 | 9.1<br>8.5 | 10 03<br>22 25 | 4.4<br>1.9 |
| 27 F | 3 00<br>13 36 | 7.3<br>10.4 | 7 50<br>21 47 | 5.2<br>0.3 | 27 Su | 3 57<br>14 20 | 8.0<br>9.4 | 9 11<br>22 20 | 5.5<br>0.7 | 27 W | 4 43<br>16 28 | 9.2<br>7.6 | 11 05<br>23 04 | 4.2<br>2.8 |
| 28 Sa | 4 12<br>14 20 | 7.0<br>9.6 | 8 51<br>22 47 | 5.5<br>0.7 | 28 M | 4 54<br>15 20 | 8.1<br>8.5 | 10 19<br>23 10 | 5.3<br>1.4 | 28 Th | 5 32<br>18 12 | 9.2<br>6.9 | 12 18<br>23 47 | 3.8<br>3.7 |
| 29 Su | 5 45<br>15 19 | 7.0<br>8.6 | 10 12<br>23 52 | 5.8<br>1.1 | 29 Tu | 5 55<br>16 52 | 8.3<br>7.6 | 11 34<br>. .. | 4.9<br>. . | 29 F | 6 25<br>19 51 | 9.4<br>6.8 | . ..<br>13 38 | . .<br>3.3 |
| 30 M | 7 03<br>17 03 | 7.5<br>7.7 | 11 45<br>. .. | 5.5<br>. . | 30 W | 6 49<br>18 50 | 8.7<br>7.2 | 0 00<br>12 53 | 2.1<br>4.2 | 30 Sa | 7 20<br>21 03 | 9.7<br>7.1 | 0 35<br>14 55 | 4.4<br>2.6 |
| | | | | | 31 Th | 7 34<br>20 11 | 9.2<br>7.4 | 0 52<br>14 09 | 2.8<br>3.3 | | | | | |

Time meridian 45° E. The hours of the day are numbered consecutively from $0^h$ (midnight) to $23^h$ (11 00 p.m.). $12^h$ is noon. All hours greater than 12 are in the afternoon (p.m.).

Heights are reckoned from the datum of soundings on the largest scale charts of the locality which is about 2 feet below mean low water springs.

FIGURE VIB-9

| Day | MARCH | | | | | | APRIL | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Slack; Flood Begins | Maximum Flood | | Slack; Ebb Begins | Maximum Ebb | | Slack; Flood Begins | Maximum Flood | | Slack; Ebb Begins | Maximum Ebb | |
| | Time | Time | Velocity | Time | Time | Velocity | Time | Time | Velocity | Time | Time | Velocity |
| | h. m. | h. m. | kn. | h. m. | h. m. | kn. | h. m. | h. m. | kn. | h. m. | h. m. | kn. |
| 1 M | 10 19<br>22 10 | 0 36<br>13 09 | 6.3<br>5.7 | 3 32<br>16 04 | 6 52<br>19 07 | 6.0<br>4.9 | 1 11 55<br>Th 23 22 | 1 30<br>14 30 | 4.3<br>3.9 | 3 57<br>17 50 | 7 43<br>20 44 | 4.3<br>2.0 |
| 2 Tu | 11 21<br>22 56 | 1 21<br>14 03 | 5.6<br>4.9 | 4 10<br>17 02 | 7 42<br>20 00 | 5.4<br>3.8 | 2 13 42<br>F . . . | 2 41<br>15 55 | 3.5<br>3.5 | 5 04<br>21 15 | 9 47<br>. . . | 3.8<br>. . |
| 3 W | 12 43<br>. . . | 2 13<br>15 09 | 4.8<br>4.2 | 4 58<br>18 48 | 8 48<br>21 26 | 4.7<br>2.6 | 3 2 02<br>Sa 14 59 | 4 08<br>17 33 | 3.2<br>4.0 | 7 27<br>22 02 | 0 12<br>12 05 | 2.5<br>4.3 |
| 4 Th | 0 15<br>14 14 | 3 17<br>16 29 | 4.1<br>3.8 | 6 06<br>21 19 | 10 30<br>. . . | 4.4<br>. . | 4 3 17<br>Su 15 50 | 5 34<br>18 35 | 3.9<br>4.8 | 9 05<br>22 30 | 1 01<br>12 55 | 3.7<br>5.0 |
| 5 F | 2 08<br>15 27 | 4 33<br>18 01 | 3.7<br>4.1 | 7 51<br>22 30 | 0 06<br>12 18 | 2.5<br>4.7 | 5 4 03<br>M 16 28 | 6 31<br>19 13 | 4.7<br>5.5 | 9 53<br>22 54 | 1 32<br>13 27 | 4.5<br>5.6 |
| 6 Sa | 3 28<br>16 19 | 5 51<br>19 08 | 4.0<br>4.8 | 9 17<br>23 06 | 1 19<br>13 16 | 3.5<br>5.2 | 6 4 38<br>Tu 17 02 | 7 12<br>19 45 | 5.5<br>6.1 | 10 28<br>23 16 | 1 56<br>13 54 | 5.1<br>6.1 |
| 7 Su | 4 23<br>17 01 | 6 52<br>19 46 | 4.6<br>5.4 | 10 11<br>23 34 | 1 59<br>13 55 | 4.2<br>5.7 | 7 5 07<br>W 17 32 | 7 48<br>20 15 | 6.1<br>6.5 | 10 56<br>23 36 | 2 14<br>14 19 | 5.5<br>6.5 |
| 8 M | 5 04<br>17 35 | 7 37<br>20 21 | 5.2<br>6.0 | 10 49<br>23 59 | 2 28<br>14 25 | 4.7<br>6.1 | 8 5 34<br>Th 18 00 | 8 20<br>20 45 | 6.5<br>6.8 | 11 23<br>23 55 | 2 33<br>14 44 | 5.9<br>6.7 |
| 9 Tu | 5 38<br>18 06 | 8 15<br>20 49 | 5.7<br>6.4 | 11 21<br>. . . | 2 51<br>14 49 | 5.2<br>6.4 | 9 5 55<br>F 18 24 | 8 52<br>21 16 | 6.9<br>6.8 | 11 50<br>. . . | 2 55<br>15 10 | 6.1<br>6.9 |
| 10 W | 6 06<br>18 34 | 8 49<br>21 19 | 6.1<br>6.7 | 0 20<br>11 49 | 3 11<br>15 14 | 5.5<br>6.6 | 10 6 17<br>Sa 18 53 | 9 25<br>21 47 | 7.1<br>6.8 | 0 14<br>12 19 | 3 18<br>15 35 | 6.3<br>6.9 |
| 11 Th | 6 30<br>19 00 | 9 22<br>21 50 | 6.4<br>6.8 | 0 40<br>12 14 | 3 33<br>15 40 | 5.7<br>6.8 | 11 6 41<br>Su 19 20 | 9 57<br>22 20 | 7.2<br>6.6 | 0 36<br>12 52 | 3 42<br>16 04 | 6.4<br>6.8 |
| 12 F | 6 52<br>19 26 | 9 55<br>22 21 | 6.6<br>6.8 | 0 58<br>12 43 | 3 56<br>16 06 | 5.9<br>6.8 | 12 7 09<br>M 19 51 | 10 32<br>22 53 | 7.1<br>6.2 | 1 01<br>13 27 | 4 08<br>16 35 | 6.5<br>6.6 |
| 13 Sa | 7 14<br>19 52 | 10 26<br>22 53 | 6.7<br>6.6 | 1 19<br>13 13 | 4 18<br>16 33 | 6.0<br>6.8 | 13 7 42<br>Tu 20 27 | 11 11<br>23 33 | 6.9<br>5.7 | 1 30<br>14 08 | 4 35<br>17 10 | 6.4<br>6.2 |
| 14 Su | 7 39<br>20 21 | 11 00<br>23 26 | 6.7<br>6.3 | 1 43<br>13 47 | 4 42<br>17 00 | 6.0<br>6.6 | 14 8 23<br>W 21 12 | 11 57<br>. . . | 6.4<br>. . | 2 04<br>14 56 | 5 08<br>17 54 | 6.1<br>5.5 |
| 15 M | 8 09<br>20 53 | 11 36<br>. . . | 6.5<br>. . | 2 09<br>14 25 | 5 08<br>17 32 | 6.0<br>6.3 | 15 9 15<br>Th 22 18 | 0 24<br>12 58 | 5.1<br>5.9 | 2 44<br>15 56 | 5 51<br>18 55 | 5.6<br>4.7 |
| 16 Tu | 8 46<br>21 33 | 0 02<br>12 18 | 5.9<br>6.2 | 2 40<br>15 09 | 5 38<br>18 11 | 5.8<br>5.8 | 16 10 31<br>F . . . | 1 27<br>14 13 | 4.3<br>5.4 | 3 41<br>17 21 | 6 56<br>20 38 | 4.9<br>4.1 |
| 17 W | 9 33<br>22 27 | 0 45<br>13 10 | 5.3<br>5.8 | 3 17<br>16 04 | 6 16<br>19 01 | 5.5<br>5.1 | 17 0 18<br>Sa 12 27 | 2 52<br>15 40 | 4.0<br>5.3 | 5 05<br>19 20 | 8 58<br>22 42 | 4.4<br>4.4 |
| 18 Th | 10 40<br>23 57 | 1 43<br>14 23 | 4.6<br>5.3 | 4 06<br>17 18 | 7 11<br>20 25 | 5.0<br>4.3 | 18 1 59<br>Su 14 07 | 4 16<br>16 59 | 4.3<br>5.7 | 7 10<br>20 44 | 10 55<br>. . . | 4.9<br>. . |
| 19 F | 12 24<br>. . . | 3 00<br>15 48 | 4.1<br>5.1 | 5 15<br>19 06 | 8 49<br>22 31 | 4.5<br>4.1 | 19 3 06<br>M 15 14 | 5 32<br>18 05 | 5.2<br>6.3 | 8 46<br>21 39 | 0 01<br>12 11 | 5.3<br>5.7 |
| 20 Sa | 1 55<br>14 13 | 4 23<br>17 11 | 4.2<br>5.5 | 6 56<br>20 55 | 10 53<br>. . . | 4.8<br>. . | 20 3 54<br>Tu 16 07 | 6 32<br>18 56 | 6.1<br>6.8 | 9 48<br>22 20 | 0 50<br>13 05 | 6.1<br>6.4 |
| 21 Su | 3 15<br>15 29 | 5 41<br>18 22 | 5.0<br>6.2 | 8 41<br>22 00 | 0 07<br>12 18 | 4.8<br>5.6 | 21 4 37<br>W 16 52 | 7 21<br>19 40 | 6.8<br>7.2 | 10 37<br>22 57 | 1 33<br>13 48 | 6.7<br>6.9 |
| 22 M | 4 11<br>16 26 | 6 44<br>19 19 | 5.8<br>6.9 | 9 52<br>22 47 | 1 08<br>13 18 | 5.7<br>6.4 | 22 5 16<br>Th 17 31 | 8 04<br>20 19 | 7.3<br>7.4 | 11 19<br>23 29 | 2 10<br>14 27 | 7.2<br>7.1 |
| 23 Tu | 4 57<br>17 13 | 7 38<br>20 06 | 6.6<br>7.4 | 10 46<br>23 27 | 1 54<br>14 05 | 6.4<br>7.0 | 23 5 54<br>F 18 08 | 8 44<br>20 57 | 7.5<br>7.3 | 12 00<br>. . . | 2 43<br>15 02 | 7.4<br>7.0 |
| 24 W | 5 38<br>17 56 | 8 24<br>20 48 | 7.2<br>7.6 | 11 32<br>. . . | 2 34<br>14 47 | 6.9<br>7.3 | 24 6 30<br>Sa 18 40 | 9 22<br>21 32 | 7.5<br>7.1 | 0 01<br>12 35 | 3 16<br>15 36 | 7.4<br>6.7 |
| 25 Th | 6 18<br>18 35 | 9 06<br>21 27 | 7.5<br>7.7 | 0 03<br>12 13 | 3 11<br>15 26 | 7.3<br>7.4 | 25 7 05<br>Su 19 10 | 9 59<br>22 07 | 7.3<br>6.8 | 0 30<br>13 11 | 3 48<br>16 09 | 7.2<br>6.3 |
| 26 F | 6 55<br>19 12 | 9 46<br>22 03 | 7.6<br>7.5 | 0 36<br>12 53 | 3 46<br>16 02 | 7.4<br>7.3 | 26 7 40<br>M 19 38 | 10 37<br>22 42 | 6.8<br>6.2 | 0 58<br>13 44 | 4 19<br>16 39 | 6.9<br>5.8 |
| 27 Sa | 7 33<br>19 45 | 10 26<br>22 40 | 7.4<br>7.2 | 1 08<br>13 31 | 4 20<br>16 37 | 7.3<br>6.9 | 27 8 16<br>Tu 20 06 | 11 17<br>23 20 | 6.2<br>5.6 | 1 26<br>14 18 | 4 50<br>17 11 | 6.5<br>5.0 |
| 28 Su | 8 10<br>20 16 | 11 05<br>23 17 | 7.0<br>6.7 | 1 39<br>14 08 | 4 54<br>17 11 | 7.0<br>6.3 | 28 8 56<br>W 20 37 | 12 01<br>. . . | 5.5<br>. . | 1 57<br>14 55 | 5 22<br>17 47 | 5.9<br>4.2 |
| 29 M | 8 49<br>20 46 | 11 46<br>23 56 | 6.4<br>6.0 | 2 09<br>14 45 | 5 28<br>17 44 | 6.6<br>5.5 | 29 9 47<br>Th 21 19 | 0 02<br>12 53 | 4.8<br>4.7 | 2 32<br>15 46 | 5 56<br>18 32 | 5.1<br>3.1 |
| 30 Tu | 9 31<br>21 15 | 12 30<br>. . . | 5.6<br>. . | 2 40<br>15 24 | 6 03<br>18 18 | 6.0<br>4.6 | 30 11 08<br>F 23 25 | 0 58<br>13 58 | 4.0<br>4.1 | 3 15<br>17 22 | 6 49<br>20 20 | 4.3<br>2.1 |
| 31 W | 10 26<br>21 51 | 0 38<br>13 22 | 5.2<br>4.7 | 3 14<br>16 13 | 6 43<br>19 03 | 5.2<br>3.4 | | | | | | |

Time meridian 128° W. The hours of the day are numbered consecutively from 0h (midnight) to 23h (11 00 p. m.). 12h is noon. All hours greater than 12 are in the afternoon (p. m.).

On the flood the current sets northeastward; on the ebb, southwestward.



FIGURE VIB-10

# MANUAL OF AMPHIBIOUS OCEANOGRAPHY

## VI INTELLIGENCE FOR AMPHIBIOUS OPERATIONS

## C. AERIAL PHOTOGRAPHY

BY

R. C. CROOKE

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# AERIAL PHOTOGRAPHY

## 1. General Information

a. Introduction: (References VI C-1, 2 and 5)
Recent studies have cast doubt on earlier enthusiastic reports on the accuracy of aerial photography for quantitatively determining underwater beach profiles. Even so, this method has proven merits, and should be used to its fullest advantage.

The working techniques presented in this section are not cut and dried. The interpreter must always use his common sense. He must have considerable knowledge of the subject to properly interpret the photographs, and should always check the data obtained by other means whenever possible.

Before selecting the method for calculating depths, studies should be made of the general terrain, the beach, and the wave pattern.

(1) Evaluation of Terrain Features: A general study of the area under consideration should be made first - including a study of all available photographs, terrain maps and hydrographic charts. An examination of the general terrain features will almost always give some indication of what to expect of the sea bed. For example:

- If the hinterland is flat and sandy or marshy, it is reasonable to assume that the sea bottom close to shore will also be fairly flat.

- If the beach is on a long stretch of regular coastline, it will often have one or more sand bars offshore.

- If large rock outcrops are present along the beach or close inland, it is possible that the sea bed will also have rock outcrops.

- If beaches are backed by cliffs or steeply rising hills, they will generally have a fairly steep underwater gradient.

- Also - coral forms follow well-known patterns, and their presence should not be difficult to predict.

(2) Evaluation of the Beach: The beach itself also supplies clues to the underwater contours. The form of the beach is determined by the material of which it is composed, by the wave climate, and by the hydrography and the topography. A wide, flat beach generally indicates a gently sloping bottom offshore, and a sharp, narrow beach often indicates a steep slope. Fine sand beaches will have slight slopes, and beaches of shale or cobble will be steep, with coarse sand beaches in between. It must be remembered that the entire beach profile is mobile, varying with the wave conditions which act on it.

(3) Evaluation of Wave Pattern: The wave pattern is roughly a reflection of the bottom contours, so a careful study of it will give a general idea of the sea bed. The following procedure is recommended:

- All photographs covering the area should be laid out on a large table, and bottom peculiarities (indicated by the wave pattern) should be checked from photograph to photograph.

- Isolated breakers should be followed over as many photographs as possible: if the wave continues to break at a particular spot, a shoal area or obstacle is indicated - - if it breaks on only one or two photographs, its breaking is probably due to a local gust of wind. (It should be remembered that refraction, change in wave length, and peaking up of the waves are all indications of changing bottom contours).

- Previous sorties should be checked to determine whether or not the wave pattern indicates the same general trend.

- If calm, clear-water photographs are available, the visible sea bed should be compared with estimates made from the wave patterns. Sand bars, rocks, and obstructions can easily be seen in most still-water pictures.

(4) Choice of method of depth calculation: The best method (or methods) of depth calculation should then be selected, depending on the available photographs, and wave condition, and keeping in mind the general aspects of the sea bed as determined by the above preliminary studies.

b. Wave Refraction - An Indication of the Bottom Topography (Quoted from Reference VI C-4 Pages 5.16-5.19. For complete theory, See Section I-D.)

"Although the effect of refraction can be used for [rough]* quantitative computations of depth, it is of greatest value for a qualitative determination of the type of bottom one deals with. After some experience, one can tell at a glance whether a crest pattern is the result of a ridge, a trough, or other bottom feature.

The effect of refraction on a wave train is demonstrated in a refraction diagram which contains two groups of curves - wave crests and orthogonals, the latter being lines which everywhere run at right angles to the wave crests. Figure VI C-10 shows a refraction diagram for waves of a period of 16 seconds - advancing from the south toward part of the coast of southern California. The chart in the lower right corner of the figure demonstrates the effect of refraction over a large area. The southern part of the coast line shown in the chart is irregular, and bottom contours, as indicated by the 600, 300, 120 and 60 foot lines, are complicated. The coast line in the middle portion of the chart is straight with nearly parallel bottom contours. The position of every tenth wave crest is shown at an arbitrarily selected time. An area to the north of Oceanside, which is blocked in on the chart, is shown on a large scale in the main part of the diagram, and the position of each wave crest is entered. In both diagrams smooth bottom contours are shown. The diagrams demonstrate the decrease in wave length and the bending of the wave crests as the waves advance into shallow water. The effect of refraction begins in this case at the outer border of the continental shelf where the depth to the bottom decreases abruptly.

Figure VI C-10 also demonstrates another effect of refraction. In deep water the orthogonals are parallel and have been drawn at equal distances. As the waves advance into shallow water, the orthogonals bend toward shore and the distances between them increase; that is, the wave

crest between two orthogonals stretches. Assuming that the energy between two orthogonals remains constant, an increase in the crest length is accompanied by a decrease of energy per unit surface area, that is, a decrease in wave height. [For example,] the wave height directly north of La Jolla must be small compared to that north of Oceanside. This is in agreement with observations...

Figure VI C-11 show wave refraction around an [idealized] island, over an [idealized] trough, and over an [idealized] ridge. Wave crests and orthogonals are shown, and regions where the orthogonals converge (large wave heights) or diverge (small wave heights) are indicated by different shading. The diagrams are drawn on a non-dimensional basis by making depth contours and linear distances relative to [the deep water wave length,] $L_o$. The purpose is to indicate that refraction will be the same however large or small the island, ridge, or trough [may be.]*"

"Refraction around the island, and the island's profile is shown in Figure VI C-11a. Orthogonals converge on the exposed, and diverge on the unexposed, sides of the island. The crests bend around the island and meet on the lee side, where considerable interference takes place. To illustrate the effect of an island at a great distance from shore, Figure VI C-11a has been redrawn at a smaller scale in Figure VI C-11b. The crests intersect behind the island in such a manner that the island appears as a source of circular waves. The height of these waves along any orthogonal decreases, approximately, as the linear distance from the island. A steep island will cause lower waves than an island with gradually sloping beaches, yet, every island, no matter how small, is effective over large distances in interfering with a regular wave pattern.

Figures VI C-11c, d show the refraction of waves over a ridge and over a trough. The crests always bend in such a manner as to become parallel with the contours. The orthogonals diverge over the trough; therefore, wave heights are lower where a submarine canyon approaches the coast line. Over the ridge the orthogonals converge sharply and such convergence may be responsible for a long swell breaking over deep shoals, as is frequently reported.

Figures VI C-12-14 are reproductions of photographs of wind waves at San Clemente Island, California. The scale is 1:12,000 and the features are all small scale. It follows that these photographs may also be considered as model presentations of what happens to a long swell approaching the shores of a large land area. In figures VI C 13-15 the wave patterns in the photographs are shown schematically in order to facilitate interpretation. The effects of two small islands marked A and C in Figures VI C 13-15 are easily recognized. Island C is elongated in the direction of the crest and offers protection against the waves

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* - [ ] added by authors
Figure numbers are all different from source quoted.

on its lee side. The waves passing to the right and left remain distinct from one another for some distance behind the island. Some waves seem to break over the ridge betwwen island A and sandpit B. The refraction over a trough is similar to the refraction in the channel between island C and the main island on Figure VI C-15. Various other features in Figures VI C 12-14 are indicated on the corresponding diagrams (Figures VI C 13-15, respectively). It has been found that, with experience, much can learned about the bottom topography from a close study of photographs without the need of any computation. This is the greatest immediate value to the interpreter of a sound knowledge of the principles of refraction."

(End of Quote from Reference VI C-4)

c. Recognition of Bars:

(1) Wave action present: (Figures VI C-16-17)

A bar is indicated by one of the following: a continual breaking of waves, a reflection, a change in wave length, or by the breaking up of the initial wave into a series of smaller waves. The observer should never decide whether a bar exists on the basis of one photograph, or by observing just one wave. Under some conditions, the presence of a bar might not be suspected (i.e. when the pictures show a small surface wind chop which obscures the bottom, but which is not large enough to be affected by the change in depth over the bar). A sufficient number of sorties must be flown to obtain photographs of the given area at several tide stages and for various wave conditions.

Figure VI C-21 plots the movement of a bar with a change in wave conditions. This shows the necessity of having last-minute pictures of a landing beach in order to keep posted on beach changes.

(2) No wave action present: Shoal spots or bars can often be detected by noting the change in bottom color (Figure VI C-19). In general, lighter patches indicate shallower water. However, if the area is near a river mouth, a light area may be due to suspended material: wave action can also cause light areas by putting material into suspension (Figure VI C-20).

If the water is clear, and if there is sufficient surface and bottom control, it is advisable to fly sorties to approximate the depth of the water over the bar - using one of the stereo methods outlined in the next section ( #2 - "Depth Determination Methods"). Another method is to note any shallow spots that are uncovered during low tide. It is advisable to fly all sorties during low tide because there is then less water over the obstacles. (Figure VI C-18). Warning: watch for large tidal currents that may also carry bottom materials in suspension, thus producing light areas. It is best to observe sorties taken on both the flood and ebb tide to eliminate this chance of error.

d. Recognition of Rip Currents:

Typical rip currents through the surf are shown in Figure VI C-22. They are readily apparent from even a casual examination of the photographs. Rips may appear and disappear with changes in wave condition and tide stage. As they are a product of wave action, none will appear on photographs taken on an extremely calm day. The photo-interpreter must have up-to-date photographs, taken within at least 24 hours of the proposed time of landings.

He should also have aerials taken under conditions (for surf and tide) that are as similar as possible to those forecast for the landing time.

e. Recognition of Littoral Currents:

It is impossible to spot a littoral current itself from aerial photographs. However, if waves are breaking at an angle to the beach, there is sure to be a littoral current. The observer should also watch for beach characteristics that are produced by the current. (Figure VI C-7):

(i) Do photographs taken a week or more apart show any detectable movement of sand along the beach?

(ii) Has the position of debris on the beach changed?

The littoral current is as variable as are the waves breaking on the shore. Figures VI C 24-25 and 26 show some results of littoral currents. From these photographs it is obvious that the currents are complex, and not just a simple matter of waves breaking at an angle to the beach.

From a statistical standpoint, one should note whether sand is accumulating at the same obstruction as this would indicate the predominant drift.

f. Recognition of Underwater Obstructions:

(1) Offshore: under certain conditions, it is possible to spot underwater obstacles. It is possible to do so when the depth of the obstruction is less than the critical depth necessary to cause waves to break (Figures VI C-27-28), and also when the obstruction is large enough to cause the waves to refract. It is advisable to view sorties at highest and lowest tides to get an approximate idea of the depth of water over the obstacle.

(2) Surf Zone: this section of the beach area must be viewed at both high and low tide. During low tide, it may be possible to locate the obstacle and to estimate its size if it is completely uncovered.

The obstacles in slightly deeper water may be overlooked entirely if on all sorties - the breaker action is too violent and if the area is always covered by foam lines.

2. Methods Available to Interpreters for Beach Profile Determination (Quoted from Reference VI C-1)

a. Introduction:

"On March 29, the Committee on Amphibious Operations sponsored a working conference of representatives of the three military services to review and evaluate methods of underwater depth determination by aerial photography for the purpose of identifying those critical problems requiring solution before the best of the several methods could be made available for general service use. As a result of the discussions, which have been separately reported, general agreement was reached on several points.

(1) All existing methods are severely restricted in their usefulness because of their opportunistic dependence upon an ideal combination of meteorological, oceanographic, and time factors and because of the skills required both in the actual operation and the subsequent interpretation.

(2) In the hands of a highly skilled team, all methods might be found useful in varying degrees as special tools to be employed in widely varying situations.

(3) Insufficient data and experience have been gained in the use of the many methods now described in manuals of doctrine to permit a valid comparison and evaluation.

(4) The difficult techniques involved and the need for experienced, skilled judgment in the planning, conduct, and interpretation of these specialized missions, suggest the formation of a special reconnaissance and intelligence group available to all commands within a single combat theater for major missions of a critical nature.

On the basis of the evidence from these discussions, the Committee urges that an effort be made to expedite the current program of field evaluation now assigned primarily to the U. S. Navy Photo Interpretation Center in Anacostia. Inasmuch as skilled pilots and photographers are readily available from the Photo Interpretation Center, many delays could be avoided by having suitable operational aircraft and flight crews assigned from the Anacostia Air Station where they would be in relative readiness to take advantage of the comparatively rare combination of wind, weather, and ocean required for critical comparison of methods...

The requirements for obtaining information from which profiles or bottom contours of enemy held shorelines can be made are becoming more rigorous daily. The methods which were considered acceptable in World War II have been largely obviated by the types of defenses now available to our enemies. No leisurely low altitude photographic missions seem to be in prospect. New types of planes and cameras built to get in, take pictures as fast and as safely as possible, and get home again eliminate some of the present methods. Other methods on this list violate the first rule of intelligence and give away our intentions to the enemy. Finally, some of the methods are open to objection because they make only spot checks (with very little control over the location of those spots) or because they are highly impracticable."

b. Depth Determination Methods

Present Methods

(1) Wave length method
(2) Wave velocity method
(3) Waterline method
* (4) Stereo - standard cameras
* (5) Stereo - Sonne Strip



* Require calm, transparent water.

Methods that have been suggested

* (6) Double-camera stereo
* (7) Oblique - vertical
* (8) Densitometer ( Spectrograph)
(9) Microphotometer (to identify waves for wave method)
* (10) Reflection
* (11) Water absorption (transparency)
(12) Wave Refraction

For Spot depths only (13-18)

(13) Color sounding
(14) Bomb, Radio, or Marker Floats
(15) Explosion Plumes
* (16) Shadow sounding
(17) Underwater explosion
* (18) Wing Shadow

c. Detailed Outline of Depth Determination Methods: (All of Section c quoted from Reference VI C-1)

*(1) Wave Length Method:

Principle
Wave lengths become shorter as waves enter shallow water. From measurements of the change in length as a wave approaches a shore, the depth can be calculated.

Accuracy
Under ideal conditions ± 3 ft.**

Requirements
Visible wave pattern; wave periods of 5 seconds or more.
Standard aerial cameras.
Scale of 1:6000 to 1:12000
Maximum overlap, recycling at least every 3 seconds.
Trained personnel for analysis.

Advantages
Standard photographic requirements.
Usable in opaque waters.

Disadvantages
Requires special wave conditions.
Useless on well-protected or very steep beaches.
Presence of small boat obstacles may not be detected.
[Unusable inside breaker zone]**

This method was used operationally at both Sicily and Hollandia and probably is the most likely of any of the methods to be used again. Its requirements are somewhat less rigorous and it uses photos which may be

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* Require calm, transparent water.
** [ ] Authors addition

"taken anyway for other purposes. It is still under test by P.I.C. and the University of California. Accurate consistent results cannot be obtained and the complexities of having a series of variables each exactly right make operational failures likely."

"(2) Wave Velocity Method:

Principle
As waves enter shallow water they slow down; from measurements of the velocity, the depth can be calculated.

Accuracy
Under ideal conditions ± 3 ft.*
Requirements
Visible wave pattern; wave with periods of at least 5 seconds.
Standard aerial cameras - adapted with devices for timing interval to 1/20 sec.
Scale: 1:6000 to 1:12000
Maximum overlap, recycling at least every 3 seconds.
Trained personnel for analysis.
Advantages
Usable wherever 5 sec. or greater waves are present.
Disadvantages
Requires special timing equipment.
Useless on steep or well protected beaches.

The absence of suitable timing devices in the field have probably kept this method from being used more often. Wright Field now has a timing clock which will fit into standard cameras to give accurate time. In tests at Camp Pendleton, Va., by P.I.C. errors ranged from 1.6 to 3.0 feet and the method is apparently unreliable as now used. Errors due to winds which apparently alter the speed of waves were observed (offshore wind slowed waves). This method is still under test with the wave length method at Cape Cod and elsewhere."

"(3) Waterline Method:

Principle
As the sea level rises and falls with the tide its waterline on the beach is a series of contour lines. By taking photos at various tide stages, the slopes between high and low water can be worked out.
Accuracy
Under ideal conditions ± 1½ ft.
Requirements
Several (6) sorties in a single tidal period, spaced from predicted tides.

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* - Addition from Reference VI C-2

Scale 1:6000 to 1:12000.
Accurate tidal predictions.
Tide range of at least 6 feet.
Bottom slope of 1:15 or less.
Standard equipment.
Personnel familiar with beaches.

Advantages
Standard photography.
Works with calm opaque sea.

Disadvantages
Gives data on limited zone.
Several sorties required.
Limited use.
Weather tides and surf surging cause inaccuracies.

This method has been used operationally against Normandy and Kyushu and checks indicate that it was fairly successful. Gently sloping beaches and quiet water are of greater importance than larger tidal range. (The criterion is slope rather than depth). The accuracy is limited by, no more so than for other methods, the measurement of the photography and its definition. Field personnel can get acceptable accuracy if the time and tide information is accurate. Any system for averaging uprushes and backrushes is all right if used throughout the process."

"(4) Wave Refraction Method:

Information Obtained and Accuracy
Bottom profiles and contours.
Accuracy not determined, depths are only approximate.

Requirements
Visible wave pattern in photographs extending from shore to deep water.
Standard camera equipment and technique.

Disadvantages
Approximate depths only
Wide coverage extending from shore to deep water required.
Useless in sheltered location.

Appears to be a qualitative method as the theory is not completely known. Is useful to show the general configuration of the bottom if the waves are the right type and visible.

(5) Stereo - Standard Cameras:

Principle
Difference in elevation between two points is determined stereoscopically by measuring the difference in parallax between the images of surface and bottom as recorded on successive photos along a flight line.

Accuracy
Under ideal conditions ± 1 ft.

"Requirements
Transparent water undisturbed by swell or large wind waves; surface rippled. Recognizable bottom features.
Experienced interpreters to operate stereocomparagraph.
Maximum possible scale (long focal length or low altitude).
Maximum overlap (fast recycling).
Standard photo equipment."

Advantages
Possible to get contours of both land and underwater.
Standard methods - wide coverage.
Photos usable for other purposes.

Disadvantages
Difficult oceanographic requirements.
Dangerous combat altitudes and speeds."

(6) Stereo - Sonne:

Principle
The difference in elevation between two points is determined stereoscopically by measuring the difference in parallax between the images of surface and bottom as recorded on a strip film.

Accuracy
Under ideal condition, ± ½ ft.

Requirements
Transparent water undisturbed by swell or large waves but with surface ruffled by wind ripples. Recognizable bottom features.
Interpreters, trained in Sonne and stereocomparagraph.
Low altitude; straight and level flight.

Advantages
Gives a picture that is the shape of the beach.
Possible complete contours of bottom.
Adaptable to new fast planes

"Disadvantages
Narrow long strip.
Difficult oceanographic requirements.
Requires use of additional high altitude photography for strip orientation.

This method was used with great success in the intelligence work for Okinawa where errors were of the order of 6" (S-7 camera). The opinion of the group was that this was a statistical accident and could not be repeated operationally. However, since these cameras are now in general use and would probably be used anyway, the method will be quite useful if proper conditions exist."

(7) Stereo - Two Camera:

Principle
Two simultaneous firing cameras in one plane give parallax.

Accuracy
Under ideal conditions to ± ½ ft.
Requirements
Transparent water. No swell or large waves but surface ruffled by ripples.
Recognizable bottom features.
Trained interpreters.
Special camera mounts.
Cameras must fire simultaneously.
Advantages
Possible accurate contouring of beach and bottom.
Usable to measure wave height if sea is rough.
Disadvantages
Special mounts and plane modifications are required.
Dangerous combat altitudes.
Difficult oceanographic requirements.

"Five spot checks at Pearl Harbor were inconclusive. Attempts to make further tests were stymied by the refusal of BuAir to allow plane modification. It was reported that the Marine Corps have tried using planes using a radio link. The results obtained are not known. Faster recycling cameras might make the method more useful. This method has never been followed up and may offer an opportunity to develop new wave techniques."

(8) Oblique - Vertical

Principle
Depth of an object is determined by measuring the parallactic displacement of its image on an oblique photograph and then calculating the difference in elevation represented by this displacement.
Accuracy
Under ideal conditions ± 2 ft.
Requirements
Calm transparent water.
Clearly recognizable bottom features.
Requires both vertical * and oblique photos. Obliques with short focal length cameras from about 200 feet; verticals at scale of 1:6000 or less.
Require special equipment including an iconoscope (oblique viewer).
Trained personnel.
Advantages
Gives depths of rocks, coral heads, boat obstacles, and contours above and below the waterline.
Disadvantages
Requires special equipment and at least two runs over the target. Hard to get the obliques to show good bottom detail.

* Original source said horizontal, corrected by authors.

Was never tried operationally. Difference of opinion as to whether it is based on valid assumptions (using iconograph)."

"(9) Densitometer (Spectrograph)

Principle
Depths are determined by comparing changes in the value of underwater areas, assuming a bottom of even tone and reflectivity. Deeper water should register darker greys.
Accuracy
Under ideal conditions ± 2 ft.
Requirements
Clear transparent water, no waves.
Sandy or even-colored bottom.
Value or contrast scales.
Standard film and cameras, but special techniques.
Personnel familiar with hydrographic problems.
Advantages
Can be used with a calm water (no ripples) and even-toned bottoms.
Disadvantages
Dangerous operational altitudes.
Limited use.
Special techniques.
Doubtful assumptions.

This is a recording spectrophotometer which was invented by Eastman Laboratories. They exerted considerable pressure to get it into general use -- apparently without success. Never used operationally.

(10) Microphotometer (not a method by itself)

This is intended as an aid in the interpretation of photos when the wave methods are used. Small changes in negative density are recorded as the negative is scanned, and waves are thus distinguished. This is an elaborate system and is intended for use in a laboratory by highly trained personnel. Members of meeting were doubtful as to its usefulness under any circumstances."

"(11) Reflection

Information obtained and Accuracy
Height of ocean swell or other wave systems with rounded wave crests.
Probable accuracy unknown.
Requirements
Lighting to produce glare area in wave system, photographic technique to photograph glare area, no whitecaps or broken water. Standard camera equipment.
Trained personnel.
Disadvantages
Limited applicability.

"Special lighting conditions requiring sun.
Possible others not yet defined.

No one present at the meeting could recall how the system worked or anything definite about it."

"(12) Water Absorption (Transparency Method)

Information Obtained and Accuracy
Shallow water depth, possible wave height under limited conditions. Depth in pools, runnels.
Probable accuracy = 10% of depth under ideal conditions.
Requirements
High sun, transparent water, good reflecting bottom, small waves.
Special camera and processing techniques.
Known water transparency.
Personnel highly trained in method.
Disadvantages
Two cameras with filter.
Difficult developing procedure.
Calibration of water transparency or knowledge of depth.
Limited applicability to areas of light bottom and transparent water.

Many reports have been written on tests of this method - all of which reported difficulties principally in the film processing. Only the original British report spoke well of the method (tested in India). Smears, scratches and developing streaks are difficult to eliminate if standard developing systems are used. Many technical problems remain which can be solved; but because of the difficulties, there is little enthusiasm about this method. May be tested further by P.I.C.

(Authors Note): Due to the conflicting opinions as to the value of this method, the following references are given for anyone interested in further investigation:
Proof Test of Water Transparency Method of Depth Determination 1948. (Reference VI C-5) and R.A.A.F. Report 1946 (Reference VI C-6.)

"(13) Shadow Sounding

Principle
Floating disks are scattered from a plane and area is photographed.
Depth of water beneath each disk is calculated from the horizontal shadow separation of the disk from its shadow on the bottom.

Accuracy
Ideal conditions - Sonne ± 1.0
Standard ± 2.0

Requirements
Calm transparent water.
Even light colored bottom tones, clear sky.
Disks must be dropped - one per sounding.

Advantages
May be used under conditions where stereo and wave methods fail.

Disadvantages
Compromises security.
Dangerous operation altitudes.
Requires special equipment.

Never used operationally, and in the tests the disks were moored. Of very doubtful value."

"(14) Color Sounding

Principle
Small containers holding colored dyes are scattered from a plane, the containers sink and release colored dyes at set depths. Photographed in color.

Accuracy
Under ideal conditions ± 1.5 ft.

Requirements
Special equipment - color sounding containers.
No swell, large waves or currents.
Color film.

Advantages
May indicate direction and magnitude of current.
Works on calm cloudy days when stereo and wave methods fail.

Disadvantages
Compromises security.
Special equipment needed which gets out of adjustment.

Tank experiments were made of dyes and pressure mechanisms but the device was apparently never field-tested. The fact that great numbers would have to be used is an added disadvantage."

"(15) Bomb, Radio, or Marker Floats

Principle
Hydrostat bomb - explodes only at predetermined depth.
Float connected to sinker line of known depth.
Radio float (modified sono-buoy) transmits signal whose frequency is controlled by depth.
Accuracy
Unknown.
Requirements
Special equipment.

These merely represent suggestions and apparently development was never started; a device, similar to item 3 is the modified sono-buoy transmitting wave meter which may be of help in determining waves for the other methods.

(16) Explosion Plumes

Information Obtained and Accuracy
Spot depths at random.
Probable accuracy unknown.
Requirements
Sunlight. Plume of water from underwater explosion.
Photograph of plume and its shadow.
Possibly special bombs. Claimed that trained personnel are not required.
Disadvantages
Bomb must explode on contact with bottom. Spot depths only.
Lighting and position of plane critical.

Originally the idea of a Britisher named Burnell. Only known report is an Admiralty publication of May 1946. Difficult to synchronize time of flight and photography with bomb burst. Probably never tested in the U. S.

A current research project at the University of California at Berkeley will publish a booklet on plume dimensions for 1 pound and 10 pound charges in terms of D/W;" [1/3 where D is the water depth in feet, and W is the change weight in pounds.]*

(17) Underwater Explosion

Information Obtained and Accuracy
Spot depths at random; accuracy poor.
Requirements
Bomb or explosion-generated waves.
Standard photographic equipment and techniques.
Disadvantages
Limited applicability, clear water.
Flight at dangerous altitudes (20-50 ft.).

* Authors' addition

Method was never used operationally - invented and tested by Colonel Philbrick at Orlando, Florida. Pilot requires special training in the method, since sun and flight direction requirements are quite rigid. Sun needs to be above 40°; bottom must be white. It is difficult to tell exactly where the plane was when the photos were taken (might require an additional camera).

(End of material quoted from the Notes by the Committee on Amphibious Operations 1951, Reference VI C-1)

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Authors' Note:

(18) Wing Shadow Method: apparently no information was available about this method as it was not discussed in the detailed outline of depth determination methods in the Committee's report.

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## 3. Operational Details on Wave Methods

"a. Theory (Reference VI C-4 Pages 5.01-5.03)*

"Waves are formed by the action of the wind on the surface of the sea. Wind waves are defined as waves which are growing in height under the influences of the wind. Swell consists of wind-generated waves which have advanced into regions of weaker winds or calms and are decreasing in height. Swell may be generated many hundreds of miles from the shores upon which it breaks. Depth determination is based upon the change in behavior of these waves as they enter shallow water.

Waves are described by their length, L, the horizontal distance from crest to crest; their height, H, the vertical distance from trough to crest; and their period T, the time interval between the appearance of successive crests in the same locality. The wave form advances one wave length during one period and the velocity of the wave is therefore, defined as

$$C = L/T \quad \text{(Equation VI C-3.1)}$$

the steepness of the wave is described by the ratio of height to length, H/L. The wave energy averaged over a wave length is proportional to the square of the wave height. Half of the energy is always present as potential energy and is associated with the wave form, whereas half is present as kinetic energy and associated with the velocity of the water particles.

The water particles do not advance with the wave but move in almost closed orbits around an equilibrium position. In deep water each water particle describes a circular path in the vertical plane, moving forward at the crests and backward at the troughs. The radii of these circles decrease rapidly with depth. (Figure VI C-29)

[For waves in very deep water the motion]** at the depth of half a wave length is only 1/23 of what it is at the surface. At the ocean bottom there can be no vertical motion, so that at depths of half the wave length or less the prevention of this vertical motion disturbs the entire character of the motion. In shallow water the paths of the particles become ellipses with their shorter axes vertical; at the bottom they merely oscillate backwards and forwards horizontally. This restriction decreases the wave velocity causing the wave length to shorten. The effect is found over an absolutely smooth bottom and is not a result of bottom friction.

The fact that the wave form advances, whereas the water particles nearly retain their position can be observed by watching a patch of foam or a floating article. Such an article does not advance with the wave but moves back and forth with the rise and fall of the water (Figure VI C-29). The forward movement exceeds the backward movement, resulting in a small net transport in the direction of wave motion. The velocity of this transport is called "mass transport velocity" and averages about $\frac{1}{2}\%$ of the wave velocity."

"Simultaneously with the decrease of wave velocity there is an increase in the fraction of the total wave energy which is carried forward with the wave form. In deep water only half the energy, the potential, is carried forward, but in very shallow water the total energy, potential and kinetic, advances with the wave form. Change in wave height is regulated by two processes, an increase in the fraction of the energy which is carried forward and a shortening of the wave. The former leads to a decrease, the latter to an increase in wave height. As the wave enters shallow water the combined result is that the wave height first decreases to about 90% of its value in deep water, then increases rapidly. The minimum height occurs at a depth of about one-tenth of the deep-water wave length. The change in wave steepness is accompanied by a change in the wave profile. In deep water waves are nearly sinusoidal in shape, but as they advance into shallow water the crests become narrower and steeper and the troughs, wider and flatter. Shortly before breaking, the waves show narrow isolated crests separated by long flat troughs. Shortening of the wave lengths and peaking up of the waves in shallow water can be readily seen on aerial photographs; either one or both provide an indication of a shoaling bottom. (Figures VI C-29-30).

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* This section quoted from Reference VI C-4, Pages 5.01-5.03 with the following exceptions:
Code numbers referring to Figures and equations are different from the source quoted. Equations have been corrected to conform with standard practive.

** Material in[ ] has been added by the authors.

Wave trains may often approach a shoreline at an angle. When this occurs the wave crests are bent or refracted, because the portion that first reaches shallow water travels more slowly than the portion which still advances in deeper water. The tendency of the wave is to refract into alignment with the bottom contours.

When bottom contours are straight and parallel to shore, the wave front will turn parallel to shore. In this case the similarity of wave refraction to light refraction is indicated by the formula

$$\text{Sin}\,\alpha_o/\sin\alpha = C_o/C$$

where $\alpha_o$ = angle which the deep water crest makes with the shoreline, $\alpha$ = angle at any point, x, in shallow water, $C_o$ = velocity in deep water, and C = velocity at point x.

The transformation of waves in shallow water therefore involves a change in wave height, length, velocity, and direction of progress. The change in height makes the waves distinctly visible near shore; from the other changes three methods of depth determination from aerial photographs have been evolved, the velocity method, the length method and the refraction method. These methods are based upon application of theoretical considerations, some of which will be briefly reviewed.

When waves of small steepness (height less than 1/200 of length) advance in water of constant depth, d, the wave length and velocity are related to depth by the equation

$$C = \sqrt{(gL/2\pi)\tanh(2\pi d/L)}$$

$$L = \frac{gT^2}{2}\tanh\frac{2\pi d}{L} \qquad d/L) \quad \text{(Equation VI C-3.2)}$$

Where g is the acceleration due to gravity and tanh is the hyperbolic tangent."

"This equation applies to both deep and shallow water. Deep water is defined as having depth exceeding one-half a wave length; the subscript, o, is used to denote deep water. In deep water the term $2\pi d/L$ in equation VI C-3.2 approaches unity and the expression becomes

$$C_o = gT/2 = 5.12T$$

(Equation VI C-3.3)

and $L_o = 5.12T^2$

where L is measured in feet, T in seconds and C in feet per second.

"The other limit of equation VI C-3.2 applies in very shallow water (where the depth is less than one twenty-fifth of the length in deep water) in which case the value of tanh $2\pi d/L$ approaches and $2\pi d/L$

$$C = \sqrt{gd} \quad \text{or} \quad L = T\sqrt{gd} \qquad \text{(Equation VI C-3.4)}$$

For purposes of depth determination the period of a wave train remains constant from deep to shallow water. Although equations VI C-3.2 - 3.4 were derived for water of constant depth they have been found valid for varying depths, with certain limitations to be given later.

In general, the wave velocity and length depend on the depth and period. Either one of these factors can be expressed by any other two according to equations VI C-3.1. The depth can be found, therefore if

(i) The velocity and length are known
(ii) The velocity and period are known
(iii) The length and period are known

The latter two relationships are most convenient and have been graphed to simplify procedure (Figures VI C-31 and 32)

Observed velocities, and therefore lengths, are generally somewhat larger than the theoretical values. This discrepancy may be accounted for by the effect of wave steepness on velocity, the increase in steepness tending to increase the velocity, other conditions being the same. Equations VI C-3.2, as stated, is accurate only for waves of steepness less than 1/200 and no satisfactory theory has been developed for steepness in deep water, $H_o/L_o$.

There is no theory which deals adequately with the breaker zone, making it necessary to place emphasis on empirical relationships. The depth of breaking depends upon the height, period of the wave, and beach slope and to a lesser extent upon wind velocity and direction..."

(End of Quote from Reference VI C-4 pages 5.01-5.03)

b. Factors Affecting Accuracy

"(1) Interpreter's Experience (Quoted from Reference VI C-4 Page 5.05)

Determination of underwater depths from photographs of waves does not require special skills on the part of the interpreters; however, the optimum accuracy can be achieved only through a basic knowledge of wave theory, of beaches, and with considerable working experience on wave photographs.

"The most important single asset is the ability to evaluate what one sees, and to recognize exceptions and local variant, that is, to know the parts of the photograph that are usable and those which must be treated with suspicion. With experience it is possible to determine the bottom topography roughly by looking at the photographs, the computations serving merely as a quantitative check. The ability to do this cannot be gained from reports and manuals. The experienced interpreter can often work to twice the accuracy of the beginner.

Coordination of activity must exist with the Photographic Officer, the Aerologist, and Hydrographer; in each case the experienced interpreter should be able to advise and, question intelligently. A cooperative relationship should exist whenever possible with engineer patrols concerned with underwater work. Perhaps most important of all, the interpreter should be familiar with amphibious strategy, tactics, and equipment.

(2) Measurement and Determination of Crest: (Reference VI C-4. p. 5.05)

"The greatest single source of error in [the mechanics of applying the] wave methods is encountered in making measurements on the photographs. Measurements should be made with extreme care: a scale divided into 0.001 feet should be used with a magnifying glass. At a scale of 1:12000 it is easily possible to make an error of 3 feet in measuring a 100 foot distance, resulting in an error of about 6% in the computed depth...

Closely allied with error in measurement is the exact determination of the crest position. The crests of low waves are always difficult to see, especially in deep water where they are broad. The tendency of the wave to peak up as it enters shallow water helps to determine a line of measurement; however, what one actually measures is the reflection from the crest and the reflection changes position relative to the crest as steepness increases. Strong winds tend to confuse crests, as do merging wave trains, refraction, reflection and local wind waves. Steep long-crested waves in shallow water, photographed with a low sun, will offer the best possibilities for crest location. In general, it seems advisable to measure to what appears to be the middle of the crest and not its front or rear part. The identification of the proper points to be measured on crests is very important and requires both patience and practice.

"(3) Wave Variability and Irregular Patterns: (Reference VI C-4 p. 5.05-5.06)

"Wave performance in shallow water theoretically conforms to a definite pattern dependent on period, depth and bottom contour. Actually there is a variance of perhaps 5% from theoretical expectations due to unpredictable local factors. Early breakers, sharply bent or broken crests, etc., should be followed from photograph to photograph over every sortie available before the conclusion is reached that a shoal area is indicated. Extremely irregular patterns often indicate that two or more wave trains are approaching the shore simultaneously.

In such cases the confusion of crest and trough makes it impossible to use the length or refraction methods. If velocity method is used, crests must be carefully identified and followed. Well developed wind waves at an angle to the swell often obscure them entirely until they peak up before breaking. With multiple wave trains, it is very easy to fall into error in determination of period.

(4) Scale Tilt and Timing: (Reference VI C-4 p. 5.07)

"A tilt of less than 2° is not an important factor; photographs indicating anything over 2° tilt should be discarded.

Known distances on the ground should be used to determine scale on each exposure whenever possible. When depending upon focal length and altitude, distances between sea level points should be measured and a relative scale determined. If the scale of a run of photographs varies within one half percent an average scale may be used. If the variation is greater, each exposure should be scaled separately and the poorest ones eliminated.

The importance of accurate timing of photographs for the wave velocity method cannot be overemphasized. Various experiments in timing were conducted, all leading to the conclusion that the only reliable timing device consists of a watch with a 10-second-sweep hand photographed on, or simultaneously with, the wave pictures. The known interval should be 1/20 second with 1/10 second being the maximum error permissible. Should this be impossible, the following expedient may be used; calibrate the standard Abrams B-3-B intervalometer with respect to voltage. Record the time between the first and last exposures with a stopwatch and determine the average interval. If the intervalometer has been carefully calibrated, the maximum deviation from this average will be about 1/5 second with about 90% of the exposures being accurate to 1/10 second. Velocity determinations should then be carefully checked against the wave length method and discarded if a consistent variation is evident."

(5) Maximum Depths: (Reference VI C-4 p. 5.07)

"Depth determinations are less accurate in deep water for two reasons. First, because in deep water a small change in wave velocity is associated with a large change in depth. Also there is a larger scatter of points due to the use of an average period. In general, it may be stated that depth exceeding one tenth of the deep-water wave length cannot be determined accurately even if the waves appear clearly on the photographs. Second, wave steepness decreases so rapidly with depth that the position of the crest either does not register on the photograph at all or no longer can be determined with sufficient accuracy. The first limitation applies to steep wind waves; the second, to a long low swell. It is usually possible in all cases to work accurately to depths of 15 feet, which is the critical zone for all landing operations."

(6) Minor Bottom Irregularities and Abrupt Slope Changes: (Reference VI C-2)

Experiments have confirmed the belief that wave depth determinations are not sensitive to minor bottom irregularities or to abrupt changes in slope. This may be partly explained by the fact that both the wave length and wave velocity methods give an average depth over distances of 75 feet or more. Whenever an irregularity is visually indicated by refraction, the particular area should be checked and double checked over as many photographs as possible. Also, whenever an irregularity or very confused wave pattern is visible it is recommended that the wave velocity method be followed (outlined in section 3 d "Working Techniques") Figure VI C-35a shows that this method gives a very good indication of the size and shape of the bar that is present. In reading the steps of this method, it will be found that there is a large chance for errors. Therefore, the method should be used only when the other velocity method does not confirm expectations.

(7) Steepness Correction: (Reference VI C-4 p. 5.07-5.08)

"Observed behavior of waves in shallow water is not in complete agreement with that implied in the classical equation. The difference may be partially explained by the effect of wave height on velocity, a factor which has not been considered in the equation. As theoretical attempts to correct for steepness have been unsuccessful, an experimental approach designed to establish empirical relations has been adopted as an immediate solution.

Experiments conducted at Scripps Institution of Oceanography have led to a corrective factor based on initial wave steepness, $H_o/L_o$. The correction, using an average deep-water wave height of 4 feet, has been incorporated into the wave velocity and wave length curves. The effect of the correction is to reduce the depths in shallow water from those computed from equation VI C-3.2 or, stated another way, for the same depth, the correction speeds-up * the velocity. The maximum correction of about one foot comes at depths of 5 to 7 feet and decreases to zero in deeper and shallower water. By applying this correction, the profiles based on computed depths are brought closer to the sounded profile and the maximum error is reduced."

(8) Evaluation of Results: (Reference VI C-2)

Recent tests on the wave methods have indicated that the results are quite unreliable. Figure VI C-35b shows the type of results obtained by using the wave methods. Figures VI C-33 and 34 show frequency distributions of depth differences. The plus (+) values indicate a calculated depth which is larger than sounded, and the minus (-) values, a calculated depth which is smaller than sounded. These three Figures show conclusively that the wave methods cannot be relied upon to even

* Author's correction. Original said "slows-down".

give results within (+) or (-) three feet under all conditions. However, the inaccuracy of these methods cannot be blamed entirely on errors in measuring and reducing the data. The most serious difficulty is in applying the theory (for waves of constant height and period) to ocean conditions -- where the waves are not uniform, either in height or period, but vary in a statistical manner. There is also the effect of crossed wave trains. Best results with these methods are expected where there is a gentle bottom and very regular smooth waves.

Enough emphasis cannot be placed on the importance of careful study of all available information. The interpreter should use all the various methods to check and double check. In field work, he should be alert for all possible aids. Before starting, he should familiarize himself with charts and Hydrographic Office Publications - particularly the Pilots. He should cooperate with UDT's and request special work of them when necessary. Lastly, he should make no calculations until forming a definite idea of what he expects to find. His report should always contain his honest opinion as to its reliability (and limitations). Otherwise, an operations officer may unknowingly take unreliable data as the truth, and plan a mission that may end in complete disaster.

To sum up, the interpreter can turn in reliable and useful reports only if he thoroughly understands the whole subject of waves, beaches, geology, and the limitations of the methods he has at his disposal.

c. Photographic Requirements: (Note: (1)-(5) are from Reference VI C-4, p.5.25)

(1) Wave and Weather Forecasting

"The success in applying wave observations to depth determination varies greatly from day to day depending on the character of the waves. Conditions are most favorable during a regular, long-crested swell. Such a swell comes from a strong, distant generating area, and may usually be predicted several days in advance with considerable accuracy. A forecast should be a prerequisite to every photographic run...."

(2) Cameras

"The K-17 camera is the best of World War II cameras for general use because of its plate size and rewind speed. The F-56 camera may be used but the seven inch plate is a disadvantage. However, the F-56 has a built-in watch which records on the film. Although this watch does not register tenths of a second, it is of advantage in determining wave period and may be replaced with a watch with a faster second hand. The K-18 camera may be used if the Army model with the Three second rewind is available; otherwise, it does not permit sufficient overlap.

(3) Altitude and Focal Length:

"... the pilot should apply the barometric pressure at the target at the time of photography to his altimeter, then correct for instrument error and temperature at the flight altitude. The aerologist should be

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consulted for an accurate pressure forecast over the target.

(4) Film and Filters:

"Any high speed black and white aerial film should give good results. (Note: best results for water surface detail and contrast in the photographs, are obtained without the use of the yellow filter on the cameras. (Reference VI C-2) Aero Kodacolor with the recommended color filters is also usable. Color film has the advantage of being particularly valuable for qualitative bottom information when the water is clear."

(5) Overlap and Interval

"An overlap of at least 80%, and an exposure interval of not over 3 seconds is required. With larger intervals, it will be difficult to identify wave crests from photograph; besides, with fewer exposures, it will not be possible to compute a sufficient number of points. One consideration must be borne in mind, that the wave velocity method is not usable unless the interval between exposures is known to at least one-tenth of a second."

(6) Photographic Light: (Reference VI C-2)

"It was found that the outlining of wave crest details was best accomplished with a low sun angle, either in the morning or afternoon, with very light wind chop on the water to give "texture" to the surface. The best time for obtaining photographic light changes with the seasons, thereby presenting a variable time for the taking of the aerial photographs. With this condition existing it is impossible to set an exact time for taking aerial photographs because it depends upon the orientation of the coast-line and the time of year. Excellent results were obtained in overcast conditions during the afternoon between the hours 1500 and 1600 during these tests.

It was found that for two nearby sites, Soldiers Club Beach, Fort Ord, California, and Tank Farm Beach, Monterey, California, the steep beach and the normally poor sun angle (in regard to the orientation of the coastline at this time of year (February and March) for this location resulted in poor photographs from a depth determination stand point. However, when a light breeze stirred up small chop on the surface, fair photographs could be obtained."

(7) Wave Direction: (Reference VI C-2)

"The direction of sun's rays in relation to the movement of wave crests is very important. Maximum wave crest detail was obtained when the sun's rays were across the crests and not parallel or down the troughs. An angle of 60° to the right or left of the perpendicular provided the best results."



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(8) Flight Lines and Tilt: (Reference VI C-4, p. 5.26)

"The flight line should be parallel with the coast and directly down the beach or waterline so that the coverage will be approximately half land and half water. A single run will sometimes be adequate, but a second and third should be made for check purposes when possible.

Flight precision is required on the part of the pilot. The plane should be kept level, and once the run is commenced, the altitude should be rigidly maintained. The photographer should be particularly careful to keep the cameras level."

(9) Time Recording: (Reference VI C-4, p. 5.26-5.27)

"The exact time of day of the run should be noted as this is essential in making tide corrections.

"The wave velocity method requires timing of individual exposures to at least one-tenth of a second. Other exposures should be timed to half-second intervals so that period may be determined; otherwise, a run to sea showing clearly the deep-water waves will be required.

The only successful timing devices developed so far have been ones using a watch with a second hand which revolves six times to a minute (ten-second sweep). If a ten-second-sweep watch is available of a size similar to the watch standard with the F-56 camera, it may be placed in the same mount. If the watch is too large, a field rig will have to be made to be mounted in the lens cone of the K-17 or K-18 cameras. This may be very simply made, on the same principle as the mount in the F-56. Another method using a large watch with a ten-second sweep has been developed and successfully used by the Photo Training Unit of Fleet Air Wing FOURTEEN...

If it is impossible to obtain a suitable watch, the following expedient may be resorted to. This method has been used by P. T. U., F. A. W. 14, and the following description is a quote from their report.

'The Abrams B-3-B intervalometer has been found to be uniformly accurate within 0.10 seconds, but must be calibrated with respect to voltage. Calibration may be accomplished on a test bench by putting a lamp in series with the intervalometer and a power supply whose voltage may be regulated to known values. A stop-watch that can be directly read to 0.10 second or estimated to that value is put in close proximity to the light bulb. The intervalometer is then set at the desired interval and energized at the voltage value to be used. One man reads the stop-watch to a tenth of a second each time the light flashes while another man records the time. At least twenty such readings are taken and the average interval is computed for each voltage used. This will vary slightly over or under the set interval, the difference being the correction. A card is then made for each intervalometer which shows the correction to be applied to the set interval for each voltage.

"The intervalometers used experimentally were calibrated for a voltage range of 22 to 28 volts. The voltage of the airplane's power supply should be recorded during each run in order that the intervalometer corrections may be applied."

Experience has indicated that all calibrated intervalometers will not be accurate to one-tenth of a second as claimed; errors as great as one-fifth of a second have been recorded. However, it may be said that about 90% of the exposures will be sufficiently accurate.

Once the intervalometer has been calibrated, the time between the first and last exposures must be carefully taken with a stopwatch by the photographer. The exposure interval is then calculated by dividing the elapsed time by the number of exposures minus one."

A new intervalometer (model B-5) has recently been developed by the Automatic Electric Instrument Company of Chicago. This intervalometer is a great deal more accurate than the standard Abrams B-3B and would be an excellent solution to the problem of timing. The Army has placed an order for these instruments, and it is understood that they are being delivered at the present time."

(10) Developing and Printing: (Reference VI C-2)

High contrast or full development is recommended for both negatives and prints. Particular care should be taken to minimize shrinkage.

High contrast and maximum wave crest detail are most satisfactorily obtained with normal development in D-19 developer. Photographic prints made on semi-matte paper (preferably sonne paper) provide the best photographic prints for wave measurements.

The following recommendations should be employed to obtain the quality of prints necessary for wave measurements when using Rapid Resisto sonne paper. This procedure is for the processing of Rapid Resisto sonne paper in the Smith, Fairchild or Morse developing machines. D-72 stock solution with no dilution can be used and the presoak step can be omitted. The following cycle with the use of a 2% 1 gallon Kodalk rinsing procedure provides excellent results;

2 passes through D-72 stock solution (full strength)
1 pass through a water rinse flowing at a sufficient rate to change the water in the tank completely during the passage of a 100 ft. roll.
3 passes through the fixing bath
1 pass through the wash water
1 pass through a 2% Kodalk solution
2 passes through the final wash.

With the exception of the transfer from developing solution to rinse which should be made quickly, the roll should be permitted to drain for about 30 seconds at each transfer. This gives a complete processing cycle of 10 passes through the solution from the first immersion in the developer to the final wash. It is most advisable to limit the length of the rolls

processed to a maximum length of 100 feet. It is very important that the exhaustion maximums of the various solutions used (200 ft. roll per 5 gal. solution) be adhered to in order to obtain high contrast and wave crest detail.

d. Working Technique:

(1) General Steps: (Reference VI C-4, p. 5.29-5.30)

(a) Determination of Tide Stage - See Section I-L
(b) Scale Determination

"If a reliable map or chart is available, or if there is a known dimension on the ground, use it to determine scale. Altitude and focal length should be relied upon only when absolutely necessary. In any case, the relative scale of a run of photographs is more important than the absolute scale. A well defined distance on the ground, preferably at sea level, should be accurately scaled on one photograph. This distance should then be used as a known length to scale the remaining photographs. The average scale of a run may be used if the scales of individual photographs do not vary more than ½% from one another. The use of average scale is simply an expedient to quicken calculations. Besides, it makes possible direct measurement of wave advance on acetate, the increased accuracy of which more than compensates for the error due to scale differences.

(c) Establishing a Base Reference Line:

There must be a line of reference established on the photographs from which all off-shore soundings are positioned. The waterline cannot be used for that purpose as its location changes with tide and with wave action. The line should be located so that it may be transferred easily from photograph to photograph, so that soundings from different photographs on one sortie or from other sorties, can be easily compared and plotted. Choose two clearly defined, but small, points at least 1000 feet apart on, or just behind the beach. The points should be at approximately the same elevation and should be as near to sea level as possible. They should be located so that a line through them will be, as nearly as possible, parallel to the water line. The line should be drawn on the photographs as a very fine ink line, projecting it at either end as necessary to cover the stretch of beach under study.

(d) Determination of Period:

(i) When the time interval between photographs is known: Select two photographs with a time interval between them of about six seconds and scale them. Select any clearly defined crest outside the breaker zone and measure its advance from the first photograph to the second. This may be done with acetate as outlined in section (d(2)). Another method is to draw a line perpendicular to the crest through two well defined land points which may be accurately positioned on both photographs. From one of the land points, measure to the crest on each photograph;

"the difference will be the wave advance. Divide the wave advance in feet by the time interval between the photographs in seconds to determine the wave velocity. Next, measure the wave length on either side of this crest on both photos and determine an average wave length. Divide the wave velocity into the average wave length; the result will be the period, T.

The period should be determined in this way at least three times and the answers averaged. There should be reasonably close agreement between the several answers.

(ii) When the time interval between photographs is known:
A less rigorous, but adequate, method of determining period may be used when the time interval between exposures is three seconds or less. Select a well defined crest which appears clearly on several successive pictures. On the first photograph, draw a line perpendicular to the crest through two points on shore; transfer this line accurately to the other photographs. Take a pair of dividers and set them on the first shore point and on the selected crest. Using this divider setting, mark the distance on successive photographs. When the next or following wave crest reaches this point on a subsequent photograph, the period equals the time interval between the first and this photograph. If the following crest does not reach the exact position of the original crest in any of the subsequent pictures, select the photographs on which it is nearest and interpolate to determine the period. For example; suppose the following crest on the fourth picture has not reached the position of the original crest on the first picture, but has passed it on the fifth. If the crest is equidistant from the point on the fourth and fifth photographs, the period is then halfway between 9 and 12, or $10\frac{1}{2}$ seconds.

(iii) When a spot depth is accurately known:

When a single accurate depth is known, or when a reliable chart is available, the period may be found by using Figure VI C-32 the curves relating period and length to depth. Select a well defined wave length over known depth on several photographs, and, using the average length, read the period from Figure VI C-32 as before.

(iv) When deep water is shown on the photographs:
The relation between wave length and period in deep water may be expressed as

$$L_o \cong 5.12\ T^2 \text{ or } T \cong \sqrt{L_o/5.12}$$

length in deep water, $L_o$, (in feet) may be measured from the photographs.

It must be remembered that this equation is valid only when the depth is greater than one-half the wave length. Deep water may be determined from chart readings or from observation of the waves. If the wave length remains constant, that is, the waves are not shortening up, the water is deep. Also, if the angle of the waves to the shoreline remains constant, the water is deep.

Make at least ten measurements of deep water wave length and average them before computing the period."

(2) Wave Velocity - Procedure: (Reference VI C-4 pp. 5.30-5.31)

(i) Scale the photographs as accurately as possible and discard those that are unusable, Figure VI C-37.

(ii) Determine the period, T.

(iii) Locate two clearly defined points along the shore as close to sea level as possible so that a line drawn thru them will be parallel to the shoreline. Circle both these points and draw a fine line through them. Locate and mark a third point as close to the same elevation as possible and forming a triangle with the first two points, accurately transfer these same points to all other photographs on which they are shown.

(iv) Place a sheet of thin acetate (preferably 0.01 inch) over the first photograph to be used and tape it down with scotch tape. Mark the three control points and the baseline. Draw lines perpendicular to the base line wherever profiles of the bottom are to be taken. It is perhaps best to draw lines at regular intervals, say, 200 feet apart.

(v) Along these range lines trace on the acetate, with a very fine line the crests of the waves, using a steel stylus or other sharp pointed instrument. The crest marks should be numbered for identification. Use only crests which are clearly defined - do not try to invent crest positions.

(vi) Remove the acetate sheet from the first photograph and secure it to the second, using the control points to accurately determine its proper position. If the points do not match closely, discard the photograph and use another. The time interval between photographs used should be about six seconds; it is usually necessary to work with every other photograph. Trace the new position of each wave crest onto the acetate, being careful to identify the proper wave in each case. The new crests should be marked to identify the proper wave in each case. The new crests

"should be marked to identify them with their original position. This may be done by lettering or, best, by connecting the ends of the two etched lines which indicate the advance of a single crest.

(vii) Remove the acetate sheet from the photograph, place it over a sheet of black paper, and measure carefully the advance of each wave crest along each range line. If the crests are at an angle to the range line, measure normal to the crests. Mark the center of the wave advance on the acetate and measure the distance of this point to the baseline. Enter these measurements on the data sheet as shown in the example. Convert these measurements into feet, using the average scale of the two photographs.

(viii) Determine wave velocity for each advance by dividing the advance in feet by the time interval between the two photographs in seconds. Enter the velocity on the data sheet.

(ix) Using the graph (VI C-31), determine the depth from the velocity and the period. Enter the depths on the data sheet.

(x) Select other photographs showing the same area, and, preparing a new sheet of acetate, work points on the same range lines. Use all the photographs in the run that cover the area, with a separate sheet of acetate for every two photographs.

(xi) Using the standard tide tables compute the level of the tide at the time of photography. Subtract this tide level from the computed depths so that depths are expressed at chart datum.

(xii) Plot at least 20 points for each range line and draw the average profile. Several range lines may be plotted on a single sheet of graph paper by plotting one profile below the other.

(xiii) Check any bottom irregularities by taking random points between the range lines. Using the same acetate sheets, determine depths from wave length and check against the velocity determinations.

(xiv) Contour the underwater area at as large a scale as is practical..."

(3) Individual Wave Method: (Reference VI C-2)

"If the bottom contours are very irregular and the wave pattern confused it is advisable to use the following procedure with the velocity method. This procedure is less accurate from the standpoint of measuring but it will be more sensitive to bottom changes and will eliminate the use of an average period for an irregular wave pattern.

(i) Scale the photographs as accurately as possible and discard those that are unusable.

(ii) Record the time of each photograph.

(iii) Plot the necessary base line, range lines, and control points on each photograph.

(iv) Starting with the initial print, mark each identifiable wave on a given range by an appropriate number. Mark these waves in all subsequent photos by the same identification number. Be sure to indicate when a crest has been left out, either because of poor definition or because it is breaking.

(v) On each print measure accurately, using a scale graduated to 0.001 of a foot, the distance from the baseline to each wave crest. Convert the measured distances to full scale values.

(vi) From the above date tabulate, for each range, the time of each photograph and the distance from the bass line to each wave. Plot these data as shown in the upper half of (Figure VI C-35). Draw straight lines between each pair of points.

(vii) The slope of the time-distance curves for each segment of the wave advance is equal to the velocity of the wave during that interval of time. Compute the velocity C for each segment and for each wave.

(viii) The wave period T, will be determined for each segment of the time displacement curve. Measure the wave length as $L_A = L_2/2$, Figure VI C-35. Obtain the period by dividing the average L by C, $T = \frac{L}{C}$.

(x) Correct the depths for the tide stage and plot the point half way between the ends of the segment.

"Notes: Because the length is used to determine the period, it is necessary that all the crests on the range line are accounted for. If more than one wave train is present, it will be necessary to handle each train separately, and if a crest is left out because its position is in doubt or because it is breaking it will be necessary to note this in order that a faulty length is not used for the waves on each side."

(4) Wave Velocity Examples: (Reference VI C-4 pp. 5.34-5.35)

"(i) The photographs were scaled against the best available map. The scale was found to be $\frac{1}{11{,}500}$ Figure VI C-37.

"(ii) The interval between photographs was accurately timed on this sortie so velocity can be used to determine the wave period T.

The interval between consecutive photographs is 6.08 seconds.
Remembering that C = L/T (equation 1)
then T = L/C
and we may find the period (T) by measuring a wave length and its velocity.
Take wave 5 in Figure VI C-37. Measuring the wave length between crest 5 and crest 6 on each photograph, we get:

on photo 1 wave length = 380'
on photo 2 wave length = 345'
average wave length = 362.5'

Measuring the advance of crest 5 from photo 1 to photo 2 we find (Figure VI C-37) the crest has moved forward 155 ft. in 6.08 seconds: 155/6.08 = 25.5 ft/sec.

now using T = L/C
we have: T = 362.5/25.5
and T = 14.2 sec.

Several such determinations were made and the average value T = 14.4 sec. taken for the Wave Period.

(iii) On photograph 1, two points near the water's edge were located and a base line established through them. A third point was circled for reference, and the three points accurately transferred to all other photographs covering the area.

(iv) A sheet of acetate was placed over photograph 1 and the three reference points and base line transferred to it. Range lines, A, B, and C were drawn on the acetate where bottom profiles were required.

(v) Along the range lines wave crests were marked off and numbered.

(vi) The acetate sheet was then transferred to photograph 2, using the three control points to accurately determine its position. The new position of each wave crest was traced on the acetate, and the two crest position lines connected on each end to avoid confusion in measurement. Breaking waves were marked BKR.

(vii) We may now begin measurements. Only range line A will be worked here. Lines B and C are, of course worked in the same way.

"Taking wave 1 (This is a foam line):

$$\text{Velocity} = \frac{\text{advance}}{\text{camera interval}} = \frac{58'}{6.08 \text{ sec}} = 9.5' \text{ pr. sec.}$$

Enter this value in the "Wave Velocity" column of the data table below. Also measure the distance of the mean position of the crest from the base line. This is 171 feet. Enter the value in the "distance offshore" column of the data table.

(viii) Having already determined a wave period of 14.4 sec. (step 2) we may now enter the graph in Figure VI C-31 with a velocity of 9.5 feet sec.; read across to intersect the period curve, and reading down we find a depth of 2.5 ft. Enter this value in the "Depth from Photographs" column of the data table below.

All waves along range line A are measured and computed in the same way. Wave 3 cannot be used because it is breaking in photo 2. All values are entered in the data table as shown.

(ix) From standard tide tables the tide stage was found to be 3.5 ft. above mean lower low water at the time of photography. This value was entered in the "correction for Tide" column of the data table."

(x) Subtracting the tide correction from the depth found on the photographs we arrive at a final value for depth at M.L.L.W.

For wave 1: 2.5 ft - 3.5 ft = 1 ft above M.L.L.W.

(xi) Using the values in the last two columns of the data table plot the profile along range lane A.

For the profile shown several additional photographs were used to get as many points as possible along line A.

DATA TABLE VI C - I

Sortie No: 944 Date: 16 June 1944 Time: 1422

Scale: 1/11,500 Camera Interval: 6.08 sec. Wave Period: 14.4 sec.

| Wave Number | Wave Advance | Wave Velocity | Depth From Photographs | Tide Correction | Depth at M.L.L.W. | Distance Offshore |
|---|---|---|---|---|---|---|
| 1 | 58 ft. | 9.5 ft.sec. | 2.5 ft. | 3.5 ft. | + 1.0 ft. | 171 ft. |
| 2 | 115 ft. | 19 ft. sec. | 11.0 ft. | 3.5 ft. | 7.5 ft. | 380 ft. |
| 3 | Breakers, Not Used | | | | | |
| 4 | 105 ft. | 17.3 ft.sec. | 9.2 ft. | 3.5 ft. | 5.7 ft. | 1,020 ft. |
| 5 | 145 ft. | 24.0 ft.sec. | 18.3 ft. | 3.5 ft. | 14.8 ft. | 1,380 ft. |
| 6 | 157 ft. | 25.8 ft.sec. | 21.5 ft. | 3.5 ft. | 18.0 ft. | 1,790 ft. |

(5) Wave Length - Procedure: (Reference VI C-4, p. 5.32)

"(i) Scale the photographs as accurately as possible. (Figure VI C-36.)

(ii) Determine the period, T.

(iii) Establish a baseline along the beach from which all offshore distances can be measured.

(iv) On a piece of acetate placed over the photograph, or on the photograph, itself, draw range lines perpendicular to the base line wherever profiles of the bottom are to be taken.

(v) Mark the intersection of the wave crests with the range lines and measure the length of the waves, or, the distance from crest to crest. Wave lengths should be measured perpendicular to the wave advance and not along the range line. Measure the distance from the baseline to the mid-point or trough of each wave. Convert these measurements to feet and enter on the data sheet.

(vi) Using the graph (Figure VI C-32), determine the depth (at the mid-point of the wave) from known wave lengths and period.

(vii) Select other photographs showing the same area and determine depths on the same range lines. Use all the photographs in the run that cover the same area.

(viii) Using the standard tide tables, compute the level of the tide at the time of photography. Subtract this tide level from the computed depths, so that depths are expressed at mean lower low water.

(ix) Plot at least 20 points for each range line and draw the average profile.

(x) Check any bottom irregularities with other wave length determinations close to the area. Check depths against those calculated by any other method.

(xi) Contour the underwater area at as large a scale as is practical."

(6) Wave Length - Example: (Reference VI C-4, pp. 5.36-5.37)

"(i) The photographs were scaled against the best available map. Scale was found to $\frac{1}{11,500}$ (Figure VI C-36)

"(ii) A fairly good chart was available for this area so it was used to determine the Period "T": At a point selected on the photograph the wave length measured 430 feet. The same point was identified on the chart and a corrected depth for tide stage of 5 fathoms read off. Entering the wave length graph with 30 feet for depth and the wave length graph with 30 feet for depth and 430 feet for wave length, we find the period "T" to 14.4 seconds. Make several such determinations and take the average T. In this example the average T = 14.4 seconds.

(iii) The base line was drawn on photograph #1, through two references points (circled on the photograph above). It was then transferred to all other photographs covering the area.

(iv) The range line (or section line) A-A was drawn where a profile of the bottom was required.

(v) Wave crests were marked and numbered along the line A-A. The breaking wave was marked BKR. Since lines 1 and 2 are foam lines, they cannot be used in the wave length method. The breaker cannot be used.
Now wave lengths may be measured as follows:

Distance from crest 3 to crest 4 = .0295 feet

.0295 x 11,500 = 339 feet

Enter 339 feet in the "wave length" column of the data table. Repeat this for all waves along the range line A-A. Measure the distance offshore (from the base line) of the midpoint of each wave. Distance from midpoint of wave 3-4 to base line = .097 feet x 11,500 = 1,114 feet. Enter 1,114 feet in the "distance offshore" column of the data table below.

(vi) Using the graph (Figure VI C-32), determine the depth from the known period and wave lengths.
For wave 3-4:
Period = 14.4 second (same for all waves on photograph)
Enter the graph with the wavelength (339 feet) reading across to intersect a period value of 14.4 seconds. In this case it will be necessary to interpolate between the 14 second curve. Read off the depth of 17.5 feet at the bottom of the graph. Enter the value 17.5 in the data table under the "depth from photograph" column.

(vii) Now using photograph #2, find depths along the same range line (A-A) on this photograph. Use all the photographs that cover the area, so as to get as many depths along line A-A as possible.

(viii) From the standard tide tables, the tide stage was computed to be 3.5 feet in the "Tide Stage" column of data Table VI C-II.

"Subtracting 3.5 feet from each of the depths found from the photographs, we now arrive at the final depth based on mean lower low water level:
Depth from Photograph-tide stage = Depth at M.L.L.W.
17.5 ft. - 3.5 ft. = 14.0 ft.

(ix) Using the value in the last two columns of the data table, plot all the points on a profile graph as shown below. Draw in the average profile. On the profile below not only points computed from Photos 1 and 2 were used, but also points computed from Photos 3 and 4 (not shown here), in order to get a good average profile.

TABLE VI C-II

Sortie No.: 944 Date: 16 June 1944 Time: 1422 Scale: 1/11,500

| Wave Number | Wave Period | Wave Length | Depth From Photograph | Correction For Tide | Depth at M.L.L.W. | Distance Offshore |
|---|---|---|---|---|---|---|
| PHOTOGRAPH 1 | | | | | | |
| 3 - 4 | 14.4 Sec. | 339 ft. | 17.5 ft. | 3.5 ft. | 14.0 ft. | 1,114 ft. |
| 4 - 5 | 14.4 Sec. | 368 ft. | 21.0 ft. | 3.5 ft. | 17.5 ft. | 1,470 ft. |
| 5 - 6 | 14.4 Sec. | 414 ft. | 27.0 ft. | 3.5 ft. | 23.5 ft. | 1,880 ft. |
| 6 - 7 | 14.4 Sec. | 486 ft. | 37.8 ft. | 3.5 ft. | 34.3 ft. | 2,350 ft. |
| PHOTOGRAPH 2 | | | | | | |
| 3 - 4 | 14.4 Sec. | 316 ft. | 14.6 ft. | 3.5 ft. | 11.1 ft. | 990 ft. |
| 4 - 5 | 14.4 Sec. | 354 ft. | 19.0 ft. | 3.5 ft. | 15.5 ft. | 1,430 ft. |
| 5 - 6 | 14.4 Sec. | 409 ft. | 26.0 ft. | 3.5 ft. | 22.5 ft. | 1,730 ft. |
| 6 - 7 | 14.4 Sec. | 471 ft. | 35.9 ft. | 3.5 ft. | 32.4 ft. | 2,180 ft. |

(7) Waterline Method: (Reference VI C-4 pp. 6.01-6.03)

(a) Theory

"If the water level is known at the time of photograph, an air photograph showing the waterline gives a beach contour based on the local datum. The level can be computed from Tide Tables if the time of photography is known.

(b) Present State of Development:

The method has been employed in Great Britain for French beaches. Experimentation in this country, carried out by the Woods Hole Oceanographic Institution and the University of California, Berkeley (Reference VI C-2) over beaches with slopes of 1:10 to 1:20, have confirmed the British findings. Greater accuracy was achieved, principally because these beaches were steeper than those used by the British.

Accuracy depends primarily on the correct determination of the water level at the time of photography. Tides must be predicted by an expert.

"Variation in the water level due to weather an wave action must be approximated. This variation will cause appreciable errors when the six sorties required are spread over a period of a week or two. The best time for photography is during periods of relatively calm water. The weather will have a minimum effect on the water level, and the absence of wave action will make the waterline easier to determine.

Whenever there are tidal ranges of any magnitude, the waterline method should be applied. Even though an extensive flying program is not practical, any sortie where the time of photography is known can be used to give a beach contour which will help check or supplement depths determined by other methods. For most places in the Pacific, tidal ranges will not exceed 8 to 10 feet, so that five, or even four evenly spaced sorties will give 3 foot contours, and the flying program need not be as extensive as that required for French beaches on the channel coast, where tide ranges are more than 20 feet. In almost every case it should be possible to get pictures flown at high and low tide. These sorties will also make possible accurate determination of any underwater obstructions close enough below the surface to be uncovered at low tide.

(c) Photographic Requirements

It is obviously desirable, if an accurate contouring of the beach is to be effected, to obtain as many waterlines as possible. There are operational difficulties in doing this and the compromise has been to ask for 6 sorties so timed that the resultant contours are evenly spaced over the maximum tidal range on the beach. This is done as follows:

(i) The top and bottom contours will obviously be obtained only at high and low water during spring tides: these are critical times that may occur rarely during the hours of daylight, and must not be missed.

(ii) For timing the intermediate waterlines, a simple device may be used which saves a great deal of time and is therefore worth describing. Draw on a squared paper a portion of a sine curve as shown in Figure VI C-38. Except in unusual cases, this represents the fall and rise of a tide, but has no scale for its duration in time, or its amplitude in height; these vary from day to day and from place to place. Take two pieces of strip celluloid; graduate one in hours and the other in feet for the tidal ranges existing at the place in question.

At a given place, suppose the highest spring tide is 21.6 feet and the lowest 1.1 feet. For an even spacing of contours between these limits it will be necessary to fly when the water is at heights 5.2, 9.3, 13.4, and 17.5 feet. Further assume that on a given day low water is at 0422 hours with a height of 3.2 feet, and high water 1134 hours with a height of 19.8 feet. Now lay the time scale TT' so that 0422 is on the line AA', i.e. low water, and 1134 on BB'. Also put down height scale HH' so 3.2 is on A'B and 19.8 on RR'. We have now 2 diagonal scales set to the conditions of this particular tide. To determine the time at which the tide will be at, say, 5.2 feet, read off 5.2 on the

"height scale, run horizontally until the vertical meets the curve, then run vertically to the time scale. In the same manner, the times for 9.3, 13.4, and 17.5 follow. The assumption that the tidal curve is always a sine curve is not justified; however, whenever the tide does vary from normal it is usually possible to obtain the proper curve from special tide tables or from the Hydrographic Officer.

Photographs may be taken at any altitude, the optimum scale being about 1:12000. Films should be printed and developed for good definition of the water line. Overlap of 60% or greater is desirable.

It is important to know the precise time of photography. Photographers should record the time when the camera is switched on and off; their watches should be accurately set. The ideal arrangement is to use a camera, such as the F-56, which is fitted with an accurate timing device.

Owing to the large accidental factors which enter into tide movements, it is desirable to make all 6 sorties on a single tide. In practice this is rarely possible, because of weather and light conditions. It is possible also, that flying over one place at such prearranged intervals would give away military intentions.

(d) Working Technique

(i) Scale the photographs

(ii) Draw the waterlines on the photographs of each sortie. This is sometimes difficult when a sea is running, as waves tend to surge up the beach, the uprush confusing the position of the still waterline. The waterline should be drawn on at least three photographs covering the same area, and averaged; this mean position will probably be fairly correct.

(iii) Compute the height of the water on each sortie. This may be done from the Tide Tables, but it is best to consult a specialist who will probably be able to take variants into account.

(iv) Make a careful uncontrolled mosaic of the low water sortie.

(v) Transfer the water lines from all other sorties to the mosaic, and mark each with its proper value. This basic mosaic should be used for all sorties flown over the beach, regardless of their date and time.

(vi) Draw sections and compute gradients."

## 4. Operational Details of Stereoscopic Methods:

a. Sonne Camera: (Reference VI C-4 pp. 2.01--2.15)

"(1) Parallax

"Difference in elevation can be measured stereoscopically by measuring difference in parallax between the images of two points on a stereoscopic pair of photographs. This method has been used for many years in the preparation of topographic maps, and is now being applied to the measurement of depths below the surface of the water. The only modification of the procedure necessary is a correction of the apparent depths for the distortion caused by the refraction of the light rays as they pass from water into air.

(2) Sonne Camera

The Sonne camera photographs a continuous strip of terrain by allowing the negative to move continuously over a fixed slit at the focal plane of the camera. The speed with which the film moves is adjusted to the height and speed of the aircraft (See Table VI C-A below). The Sonne camera has been modified to permit stereo coverage by using two lenses and partitioning the camera cone so that two strips are photographed side by side on the negative. Parallax is obtained by causing the lens assembly to be rotated in a horizontal plane so that one lens is in advance of the other. As a consequence each image is recorded from two camera positions. One lens shooting forward, records an image as the plane approaches and the other lens shooting aft, records the same image after the plane passes over.

The angle that the lens assembly is rotated from a neutral position -- that in which one lens is in advance of the other -- determines the amount of parallax which can be measured for any particular height or depth.

Sonne strip photography is especially adapted to low altitude, and permits an accurate determination of height and depth. At 200 feet altitude with 10 degrees angle of view setting, the plane travels about 35 feet from the position where a particular point is recorded by the forward lens to where the same detail is recorded by the aft lens. The relatively short interval of time between exposures of the same point at high plane speeds is of particular value in depth determination in that the top surface of the water is photographed in a nearly static condition, producing a stereoscopically visible base or datum level from which to make depth measurement...

If the film is not perfectly synchronized with the altitude and plane speed, the photograph will be distorted along the line of flight. The S7 camera can be synchronized by manual adjustment as can the S5SB, or it can use an electronic scanning device for automatic synchronization. The automatic device maintains synchronization within 10%. Errors in altitude or plane speed often cause the manual synchronization to vary 50% or more. For best results the automatic control is recommended. The automatic control will probably be available in the field soon after the cameras are available.

"The S5SB model uses two four inch (100 mm) Extar lenses, exposing two $4\frac{1}{2}$ inch continuous picture strip, side by side, on standard $9\frac{1}{2}$ inch aerial film. The S7 is similar except that two (2) stereo lens cones are available (100 mm and 88 mm). A 6" single lens cone for non-stereo strip pictures is also available.

A maximum stereo lens angle rotation of 15° is possible with both models.

TABLE VI C-II
CAMERA SETTING 100 MM LENS
SONNE CAMERA MODEL S5SB

| Altitude: | Speed in Miles Per Hour | | | | | |
|---|---|---|---|---|---|---|
| (Feet): | 200 | 225 | 250 | 300 | 350 | 400 |
| 100 | 11.5 | 13.0 | 14.4 | 17.3 | 20.2 | 23.0 |
| 150 | 7.7 | 8.7 | 9.6 | 11.5 | 13.5 | 15.4 |
| 200 | 5.8 | 6.5 | 7.2 | 8.7 | 10.1 | 11.5 |
| 250 | 4.6 | 5.2 | 5.8 | 6.9 | 8.1 | 9.2 |
| 300 | 3.8 | 4.3 | 4.8 | 5.8 | 6.7 | 7.7 |
| 400 | 2.9 | 3.2 | 3.6 | 4.3 | 5.0 | 5.8 |
| 500 | 2.3 | 2.6 | 2.9 | 3.5 | 4.0 | 4.6 |
| 600 | 1.9 | 2.2 | 2.4 | 2.9 | 3.4 | 3.8 |
| 700 | 1.6 | 1.8 | 2.1 | 2.5 | 2.9 | 3.3 |
| 800 | 1.4 | 1.6 | 1.8 | 2.2 | 2.5 | 2.9 |
| 900 | 1.3 | 1.4 | 1.6 | 1.9 | 2.2 | 2.6 |

Inches of Film Per Second

Tables are supplied with each camera for the individual lens types.

(3) Derviation of Parallax and Refraction Formulae

(a) General Formula (See Figure VI C-41)

Symbols for Figure VI C-41

a = Lens angle on Sonne Stereo camera
H = Altitude of camera above datum plane
O = Any point on datum plane
A = Any point at distance below datum plane
F = Focal length of camera
P = Difference in parallax between O and A on Photo
P = Difference in parallax between O and A on datum plane.
dh/dp = Rate of change of height with respect to parallax, or height per unit parallax
h = Height

$$\frac{h}{\frac{1}{2}P} = \cot \frac{a}{2} \qquad h = \frac{P}{2} \cdot \frac{\cot a}{2}$$

$$\frac{H}{F} = \frac{P}{p} \quad , \quad \frac{pH}{F}$$

$$\text{Height } (h) = \frac{pH}{2F} \quad , \quad \frac{\cot a}{2}$$

$$\frac{dh}{dp} = \frac{H}{2F} \quad , \quad \frac{\cot a}{2}$$

$$\text{Height} = \frac{dh}{dp} \times p$$

Example

F = 100 mm
a = 10°
h = 100'
p = 3.2 mm

$$\frac{dh}{dp} = \frac{100}{2 \times 100} \quad \cot \frac{10}{2} = \frac{100}{200} \times 11.43 = 5.73 \text{ ft/mm}$$

Height = 5.73 x 3.2 = 18.4 ft.

(b) Velocity Formula (Figure VI C-41)

$\frac{H}{F} = \frac{V(p)}{V(f)}$ (Film synchronized to scale with ground speed)

V(p) = Ground speed of plane

V(f) = Film speed

Substituting $\frac{V(p)}{V(f)}$ for $\frac{H}{F}$ in general formula

$$\frac{dh}{dp} = \frac{V(p)}{2V(f)} \quad \cot \frac{a}{2} \qquad \text{Height} = \frac{dh}{dp} \times p$$

Example: V(p) = 200 mph = 293 ft-sec

V(f) = 11.7" per sec = 297 mm per sec

a = 10°

p = 3.2 mm

$$\frac{dh}{dp} = \frac{293}{2x297} \quad \cot \frac{10}{2} = .494 \times 11.43 = 5.65$$

Height = 5.65 x 3.2 = 18.1 ft.

"(c) Ratio Formula:

Symbols for (Figure VI C-42)

A = Photograph image point of object above ground
B = Photograph image point of object on ground
$D_1$ = Offset of picture image A or total parallax value for plane height above A
D = Offset of picture image B or total parallax value for plane height above B

Then, $D-D_1$ = parallax value (p) for height of A above B

$$\frac{H}{D} = \frac{h}{D-D_1} - \frac{h}{p}$$

$$h = \frac{Hp}{D}$$

$$\frac{dh}{dp} = \frac{H}{D}$$

$$\text{Height (h)} = \frac{dh}{dp} \times p$$

Example: H = 100'
D = 17.5 mm
p = 3.2 mm

$$\frac{dh}{dp} = \frac{100}{17.5} = 5.72 \text{ ft/mm}$$

Height = 5.72 x 3.2 = 18.3 ft.

(d) Depth Factor:

When the above formulae are used for underwater depth determination (with the datum plane as sea level) a correction must be made for refraction at the airwater interface. Following is the derivation and use of this correction formula.

(See Figure VI C-43)

$$ha = \frac{OQ}{\tan i}$$

$$ht = \frac{OQ}{\tan r}$$

$$\frac{ht}{ha} = \frac{\tan i}{\tan r}$$

or

$$ht = ha \frac{\tan i}{\tan r}$$

"r" is calculated from the index of refraction:

"$\frac{\sin i}{\sin r}$ = 1.336 (for air water).

"i" is calculated as follows: (See Figure VI C-44)

"a = F tan $\frac{a}{2}$ where F = focal length, a = stero lens setting b is measured on the strip film. The measurement is made from the center of either strip (line on the film representing successive points directly under the lens)

$c = \sqrt{a^2 - b^2}$ ; $\tan i = \frac{c}{F}$

there: $\tan i = \frac{\sqrt{(F \times \tan a/2)^2 - b^2}}{F}$

$\frac{\tan i}{\tan r}$ = Depth Factor (n)

See graph in Figure VI C-46 for depth factor (n).
Use of Graph in Figure VI C-46.

(i) Find distance of depth point from center line of either half of photo strip.

(ii) Spot distance value for lens setting "a" at bottom of graph.

(iii) Read up to curve and across for depth factor (n).

(4) Present State of Development

(a) Tests at Cat Cay, Bahama Islands

A test of photographic depth measurements with the Sonne S5SB aerial strip camera was made at Cat Cay, Bahama Islands during the period May 17 - 23, 1944. Conditions of the test were held as closely as possible to conditions anticipated in enemy-held areas.

The camera was mounted in an F6F3 type plane. The pilot was briefed at Morrison field, West Palm Beach, Florida and flew out to an area at Cat Cay, Bahama Islands where leadline soundings of depths ranging from 2' to 14' feet, have been made previously. The area was photographed at altitudes of 100' and 200' with plane speeds ranging from 200 to 350 miles per hour. The photographs were processed and the depth measurements made at Morrison Field photographic laboratory.

The results of 24 depth measurements showed an average error of 0.55' with a maximum error of 1.4'. The measurement errors are not of a percentage nature. A 2' depth may be measured with the same numerical error that of a 14' depth.

Conditions of this test would closely approximate conditions anticipated in Pacific coral areas regarding weather, water clarity, etc.

"(b) Tests at Cape Cod, Mass.

Another test of photographic depth measurements with the Sonne S5SB camera was made May 31, 1944 at Cape Cod, Massachusetts near Provincetown.

This area was chosen because the turbidity of the water and lack of bottom detail provided a test of the method under difficult conditions. Tidal control and soundings of a small area were provided by the Woods Hole Oceanographic Institution. Only 4 measured depths ranging from 8' to 10' could be referred to high altitude standard aerial photography because of turbidity and lack of bottom detail.

The camera was mounted in an F6F3 type plane. The pilot was briefed at Hyannis Field, Massachusetts. The area was photographed from an altitude of 200' with plane speeds varying from 250 to 300 miles per hour. The photo depths were measured at the Photographic Interpretation Center, Naval Air Station, Anacostia, D. C. The four depths measured were photographed four times.

Two operators, one experienced and one relatively inexperienced, made 28 measurements on the four depths. The average error of the experienced operator was .69'. The average error of the inexperienced operator was .75'. The maximum error was 1.8'.

(c) Conclusions

This depth determination method is useful only where the water is clear enough to allow the bottom to be photographed and where there is sufficient bottom detail to refer the depths measured on the photographic strip to high altitude small scale photographs.

With well trained personnel, under optimum conditions of water clarity and bottom detail (as in Pacific coral areas) it is felt that the errors of the stereo-strip camera method can be held consistently below the 1.4' maximum error at Cat Cay.

The results at Cape Code indicate that even under unfavorable conditions the method may prove useful.

It should be emphasized that when employing this method, good results depend upon the experience and the cooperation of the photographic, flying, and interpretation personnel. It is suggested that the flyers, photographers and interpreters be trained together to function as a unit.

(5) Photographic Requirements

(a) Conditions of Photography

--Water: The water should be clear and free of silt or other material which would obscure the bottom.

"--Bottom. The bottom should have ample detail to provide reference for measurements and for transferring the photographic sounding from the strip photograph to a chart or a small scale photograph.

--Lighting: The best water surface control with maximum bottom detail is found where the photographs are taken when the sun angle is 45° above the horizon. The time of day for a 45° sun angle for any area can be calculated from the included graphs(Appendix). The surface becomes increasingly dense and obscures the bottom as the sun approaches its zenith. If the sun is too low the photographs lack surface control. The range of the sun angle above the horizon for good photography is approximately 35° to 55° with the optimum at 45°.

--Weather: Best results are obtained in clear weather with the wind velocity ranging from 8 to 10 knots. Depth measurements should not be attempted with this method where wind velocities exceed 20 knots. Navy Aerologists are now able to forecast the sea surface conditions (height of waves, etc.) in addition to regular weather predictions. Because of the importance of this information, aerologists should be consulted in the planning of strip camera depth flights.

--Lens Angle Setting: With plane speeds above 350 mph, a lens angle of 15° should be used. For speeds of 200 to 350 mph a 10° setting is best. Below 200 mph, a 5° setting should be used. The lens angle must be diminished with slower plane speeds in order to maintain a stereoscopic water surface.

--Plane Speeds: Greater plane speeds yield greater accuracies of measurement by permitting larger lens angle value to be used (accuracy of measurement increases as the lens angle increases). For best results photographically and tactically, a fighter plane should be used. Probably the most practical speed would be the maximum obtainable in level flight.

--Altitude: Best overall results are obtained at an altitude of 200'. While accuracy is greater at lower altitudes, the field of view is smaller and the camera synchronization is harder to control.

--Plane Instruments: The plane's air speed indicator should be carefully calibrated. An absolute altimeter should be installed. This is the only altimeter which accurately gives the plane elevation above ground or water. The aneroid altimeter registers only elevation above sea level and is subject to errors due to barometric changes and variation in the plane's air

"speed. The absolute (radio) altimeter is essential for safe flying at the required altitudes.

--Flight Lines: The photo strip covers only a relatively narrow width; hence flight lines must be accurate. The maximum coverage of a single strip with the four inch lens at 200 feet altitude is only 225 feet in width. Further distortions which affect vertical measurement are brought about by change in flight direction or fore and aft tilt of the plane in flight during the time the film is being exposed.

--Exposure: With the black and white films, exposure should be on the full side in order to obtain good bottom detail. In the bright sun of the Bahama Islands this detail has been obtained on Special Aero Pan, using no filter, at 1/150 second at f11; and on Aero-Infra-Red with no filter at 1/150 second at f5.6 to f8. With Aero Kodacolor, the exposure should be "on the nose" and it is advisable to open up one-half stop for dark bottom, and close down one-half stop for light bottom.

Exposure on the Somme camera is a function of three things:

(i) slit width,
(ii) film rate of travel, and
(iii) lens aperture setting.

The slit width can be varied in steps of 0.010 inch each. The film speed can be varied up to 25 or 30 inches per second. The Ektar is an f3.5 lens, wide open. In order to estimate the exposure settings of the three factors given above, the exposure time should be calculated as follows:

$$\text{Exposure time in seconds} = \frac{\text{slit width in inches}}{\text{Film speed in inches per second}}$$

At a film rate of travel of 3.8 inches per second, and a slit width of .038 inch, the exposure time equals 0.038/3.8, or 0.01 second (1/100). A slit width of 0.025 inch has given sharp results on preliminary tests. Using this slit width and a film rate of travel of 3.8 inches per second for 300 feet altitude, and a ground speed of 200 miles per hour, the exposure time is 0.25/3.8 or 1/166 second. These settings have given correctly exposed Aero Kodacolor at approximately f8. The exposure for each ground speed and altitude can be calculated in similar fashion.

"--Films and Filters: Good surface and bottom detail may be obtained with either black and white or Aero Kodacolor films. For best bottom detail, no filter is used on the camera lens for the black and white films. With haze conditions, it is best to use the minimum correction filters required to penetrate the haze. Good results have been obtained with films such as Special Aero Pan, and Super XX. Aero-Infra-Red film, used also with no filter, gives very good water penetration due to its high blue speed.

The Sonne Camera is ideally suited for aero color films like Aero Kodacolor which give positive transparencies with field processing. The filters recommended by the manufacturer for low altitude photography are the EF (Emulsion correcting) filters, and the HF-2 (ultra violet) filter.

--Development: Development for average to above average contrast is recommended. With Special Aero Pan, 20 minutes at 68° F., and with Super XX Aero Pan, 25 minutes at 68° F, in DK-50 on the Smith Aerial Film Developer, have given satisfactory results. This gives a contrast (gamma) of approximately 1.4. Aero-Infra-Red exposed without a filter requires only 8 minutes in DK-50 or better, 10 minutes in D-76.

--Prints: The prints should have as high a contrast as is possible and still keep the highlight (surface) and shadow (bottom) detail. With a dark bottom, a compromise of lower contrast is necessary. If Aero Kodacolor is used, it may be desirable to make a duplicate negative, and a positive print by contact. In order to have correct orientation, it is necessary to make the print with the base of the dupe negative film in contact with the paper emulsion. Some loss in sharpness is caused by this method of contact printing but usable results have been obtained. Projection printing can be used for better sharpness. Duplicate Kodacolors may also be made and used as transparencies.

(b) Necessary Information Provided by the Pilot for Each Run:

Altitude
Airspeed
Date and Time
Camera settings (film speed, aperture, lens diaphragm, lens angle)
Wind velocity and direction (pilot's estimate)

A motion picture recording unit is being developed to record film speed, airspeed, altitude, and film footage during each photographic run. This unit will probably be available soon after the Sonne S7 cameras reach the field.

"(6) Working Technique

(a) Preparation of Photographs

Method of Using Fairchild Stereocomparagraph with Stero-Strip Photographs:

The Stereo-strip photograph is cut into a number of small separate photographs (either prints or transparencies) so that they may be oriented under the Fairchild stereocomparagraph in the same manner as ordinary stereo pairs. Certain of the distortions due to deviations of the plane from straight, uniform, level flight can be eliminated from the parallax measurements by the rotation of one of the photographs of the pair.

The adjustment is made in the following manner:

The floating mark is adjusted at a point on the water surface near one edge of a pair of photos. A second point at the water surface on the other edge of the pair is selected such that a line connecting these two points is perpendicular to the direction of flight. Using the first point as a center, one photograph is rotated until the second surface point appears at the same elevation as the first (by reference to the floating dot). This orientation eliminates distortions which effect parallax measurements. The photos should be reoriented for each line perpendicular to the flight direction along which parallax measurements are to be made. The stereo leveler is recommended as a convenient means of effecting this rotation...

(b) Sonne Viewer Measuring Device

This measuring device is designed to fit in the regular Sonne viewer for making vertical measurements without cutting the film. Provision is made for applying the principles of rotation. Instead of rotating the photograph, one of the floating marks can be made to travel across the film in such a manner that all surfaces of a plane (i.e., the water surface) can be given the same parallax value along any line perpendicular to the flight direction.

As when using the stereocomparagraph, one point on the water surface is used as a center of rotation. With the floating mark over the point, the proper adjustment is made to lock the instrument with this point as a center of rotation.

Then choosing another surface point on a line with the first point, perpendicular to the flight direction (without changing the parallax reading) the mark is set at the second surface point with the offset adjustment. All surface points along this line perpendicular to the line of flight should have the same parallax value if the adjustments are properly made. CARE SHOULD BE TAKEN THAT NO DEPTHS ARE MEASURED BETWEEN ANY TWO SURFACE POINTS WHICH DO NOT HAVE THE SAME PARALLAX READING.

"The viewer allows measurement to be made conveniently on the negative increasing the accuracy by eliminating possible distortions in printing. The time required for printing is also eliminated.

(c) Interpretation of Photographs

(i) Parallax Measurements: When the photos and floating mark device have been properly aligned, the operator is ready to make parallax measurements. At least five readings should be taken on each bottom point and on each surface control point. If the measurements are not in good agreement, several more readings should be taken. An accuracy of at least - .03mm at each point is possible if enough readings are taken to establish a good value. The mean reading of the bottom point is subtracted from the mean value of the surface giving a differential parallax value with an accuracy of - .06 mm.

(ii) Bottom Readings: At times, the ruffled water surface distorts the bottom detail such that consistent readings cannot be made at any one point. While individual points may be indistinct, the bottom level is usually apparent. To get consistent readings the observer must move the dot about so that it appears to ride on the bottom.

NO ATTEMPTS SHOULD BE MADE TO MEASURE DEPTHS ON PHOTOGRAPHS WHEN CONSISTENT READINGS CANNOT BE MADE.

(iii) Establishing Surface Control: Accurate water surface control is necessary to establish a datum from which to measure depths. The easiest datum to use is the actual stereo-surface of the water. A more accurate datum is established by non-moving objects upon the surface.

The movement of the water surface will affect the parallax values in some cases. Under combat conditions of photography, the time interval between the recording of the forward lens and the after lens will usually range from about .02 second to .08 second. With a 100 mm lens the picture scale will be 1mm = 22' at 200' altitude. A moving object is photographed with an untrue parallax value due to its motion between lens recordings, making the object appear in an untrue vertical position. If a wave for example is moving 3' per second it will move .06' to .24' between exposures. If the movement is parallel to the flight direction it changes the parallax values .03 mm to .12 mm at a photo scale of 1 mm = 2'. This movement causes the wave to appear .5' to 1.8' higher or lower than its actual position depending on whether the wave is moving with or opposite to the line of flight. If the wave is moving perpendicular to the line of flight, it has moved perpendicular to the direction in which the parallax is measured and does not affect the measurement. Therefore, it appears in its true vertical position.

"To eliminate errors due to wave movement between lens exposures, the line of flight should be perpendicular to the direction of water surface movement. The observer should choose the small waves (wind ripples) rather than the larger waves for surface control because wind ripples ordinarily move about 3' to 4' per second while large waves may be moving much faster. The error can usually be held to less than half a foot by applying a correction to the surface datum using an average wind ripple velocity of 3.2 per second in fairly quiet water (wind 2-8 knots), 3.5' per second in moderately ruffled water (wind 8-15 knots), and 3.9' per second in ruffled water (wind 15-20 knots). Depths should not be measured when the wind velocity exceed 20 knots.

The following formula for wave velocity can be used where the wave length can be accurately measured on the photographs:

$C_o = 5.12L_o - .6$

Where: $C_o$ = Velocity

$L_o$ = Average wave length in deep water*

Example: Wave length = 2.8'

$C = 5.12 \times 2.8 - .6$

$C = 14.4 - .6$

$C = 3.8 - .6 = 3.2$ feet per second

The component of movement along the line of flight is applied to the parallax measurement, giving a plus correction to the depth if the ripple moves with the direction of flight and a minus correction if it moves in the opposite direction.

The direction of movement can be established by the accurate pattern of the ripples pointing downwind.

COMPUTATION OF PARALLAX ERRORS DUE TO SURFACE MOVEMENTS (Figure VI C-45)

Example: Plane speed: 300 mph (440' per second)

Ripple velocity (estimated): 3.2' per second
Altitude: 200' Focal length: 100 mm
Len setting: 10°
Distance plane travels between exposures: 2 x altitude x $(\text{Tan } \frac{10^\circ}{2}) = 35'$

Time between exposures: = $\frac{\text{distance between exposures}}{\text{plane speed}} =$

$\frac{35'}{440'}$ per second = .08 second

Ripple moves between exposures: $3.2 \times .08 = .25'$

"Component of ripple movement in line of flight:
.25' x cos 60° = .12'
Scale: 1 mm = 2' Therefore .06 mm = .12'
Parallax correction: -.06 mm (minus because movement is opposite to plane direction)

The above correction is subtracted from the parallax value of the depth. Thus, if the parallax reading is .47 mm from surface to bottom it should be corrected to .41 mm.

(d) Uses of Parallax and Refraction Formulae

The term $\frac{dh}{dp}$ signifies vertical height per unit of parallax. Where parallax is measured in millimeters, $\frac{dh}{dp} = 10'$ means that one millimeter of parallax equals 10' of elevation change. The various parallax formulae included herein are for use with instruments calibrated in millimeters. For devices calibrated in units other than millimeters, the unit used should be substituted for millimeters in all of the formulae.

There are three parallax equations available for use with the strip camera; the general formula, the velocity formula, and the ratio formula.

If ground speed is known more accurately than the altitude, the velocity formula should be used.

If the altitude is more reliable than the ground speed and the autosynchronizing control is employed, the general formula should be used. If the altitude is accurately known and the synchronization is manually controlled the ratio formula should be used.

(i) General Sonne Parallax Formula

$$\frac{dh}{dp} = \frac{H}{2F} \cot \frac{a}{2} \qquad \text{Height (h)} = \frac{HP}{2F} \cot \frac{a}{2}$$

where

$\frac{dh}{dp}$ = Height per mm

H = Plane altitude (in feet)
F = Focal length (in mm)
a = Sonne lens angle setting
P = Parallax measurement (in mm)

Depth: = $h\,(n) - \frac{HP}{2F} \cot \frac{a}{2}\,(n)$

where: n = Refraction factor (see graph of "n" values, Figure VI C-46)

"The accuracy of depth measurement using this formula depends on the accuracy of altitude and the accuracy of synchronization.

(ii) Velocity Parallax Formula:

$$\frac{dh}{dp} = \frac{V(p)}{2V(f)} \cot \frac{a}{2} \qquad \text{Height } (h) = \frac{V(p)p}{2V(f)} \cot \frac{a}{2}$$

where: V(p) = Ground speed of plane (in feet per second)
V(f) = Film speed (in mm per second)
a = Lens angle setting
p = Parallax measurement (in mm)

Depth: $hp\ (n) = \frac{pV(p)}{2V(f)} \cot \frac{a}{2}\ (n)$

Where: n = Refraction factor (see graph of "n" values, Figure VI C-46)

This equation depends on the accuracy of ground speed. The ground speed errors can be minimized by flying perpendicular to the wind direction. This equation is not dependent upon accurate synchronization.

(iii) Ratio Parallax Formula:

The principle of this equation is that the ratio of the image point offset on the two strip pictures to the altitude equals the ratio of the parallax value to the vertical height of the image.

$$\frac{dh}{dp} = \frac{Hp}{D} \qquad \text{Height } (h) = \frac{Hp}{D}$$

Where : H = Plane altitude
D = Image offset (in mm)
p = Parallax measurement (in mm)

Depth = $hp\ (n) \frac{Hp}{D}\ (n)$

Where : n = Refraction factor (see graph of "n" values Figure VI C-46.)

The accuracy of depth measurements using this equation depends upon the accuracy of the altitude.

If an object of known dimensions is photographed, the true altitude can be determined by scaling the dimensions on the photographs perpendicular to the line of flight. However, unless known objects were dropped on the water surface, it would be unusual to fine enough known objects to establish sufficient altitude control.

To minimize the altitude error, an absolute (radio) altimeter should be used. This equation is not dependent upon accurate synchronization. With the automatic synchronization attachment, the value D will be a constant for any angle "a". $D = 2F \tan \frac{a}{2}$ where F equals focal length, "a" equals lens angle setting."

(End of quote from Reference VI C-4 pp. 2.01-2.15)

b. Standard Cameras: (Reference VI C-4 pp. 3.01-3.09)

"(1) Parallax

Difference in elevation can be measured stereoscopically by measuring difference in parallax between the images of two points on an overlapping pair of photographs. This method has been used for many years in the preparation of topographic maps, and is now being applied to the measurement of depths below the surface of the water. The only modification of the theoretical procedure necessary is a correction of the apparent depths for the distortion caused by the refraction of the light rays as they pass from water into air.

(2) Derivation of Parallax Formula

The relation between difference in elevation on the ground and difference in parallax on the overlapping pair of photographs is developed mathematically below: (See Figure VI C-51)

Symbols for Figure VI C-51

B = Actual Flight Base.
b = Flight Base Measured on Photo.
P = Parallax on Datum Plane Due to h.
P = Parallax on Photograph Due to h.
H = Height of any point from Datum Plane.
F = Focal Length of Camera.

(1) $\frac{B}{P} = \frac{H + h}{h}$ (2) $\frac{B}{P} = \frac{b}{p}$ (3) $\frac{b}{p} = \frac{H + h}{h}$

(4) Cross Multiplying; $hb - hp = pH$ (5) $h = \frac{pH}{b-p}$

(6) Since p is ordinarily small as compared with b, an approximate formula may be written which will be sufficiently accurate for water depth determination; $h = \frac{pH}{b}$

Then; $\frac{dh}{dp} = \frac{H}{b}$

"If the parallax is measured in millimeters, and the change in elevation is expressed in feet, dh/dp is expressed in feet per mm; therefore H is expressed in feet and b in mm.

In a case where H = 4000' and b = 91.4 mm (60% overlap, 9" x 9" plate size) $\frac{dh}{dp} = \frac{4000}{91.4} = 43.7$ ft/mm, or 1 mm of parallax on the photo represents 43.7 ft. difference in elevation on ground.

(3) Refraction Corrections:

A correction for refraction is necessary to change the apparent depth measured on the photos to the true depth. One method of correction is described below: Figure VI C-52

Symbols for Figure VI C-52

H = Altitude
F = Focal Length
$h_A$ = Apparent Depth
$h_T$ = True Depth
L = Angle of Incidence
r = Angle of refraction
S = Bottom Point.

(1) $hA = \frac{SA}{\tan i}$ (2) $hT = \frac{SA}{\tan r}$ (3) $\frac{hT}{hA} = \frac{\tan i}{\tan r}$

(4) $\frac{hT}{hA} = \frac{ST}{SA}$ (assuming Aa and Tt parallel: this relationship is accurate within 1% when depths are less than 1/10 altitude)

(5) $\frac{ST}{SA} = \frac{st}{sa}$ (6) $\frac{st}{sa} = \frac{\tan i}{\tan r}$

where angle "i" is known (computed from photograph; tan i = distance of image from center of photo divided by focal length), angle "r" can be calculated from the law of refraction $\frac{\sin i}{\sin r} = 1.336$ (for air-water)

(1) the difference in parallax between any surface point and bottom point "a" = $a_1q_1 - a_2q_2$ (Figure VI C-54)

Symbols for Figure VI C-54

a = Image of Bottom Point
s = Image of point directly over bottom
t = Position image would occupy without water refraction
$c_1$ = center of print I
$c_2$ = center of print 2

(2) Although the exact position of "s" is unknown, the ratio $\frac{s_1t_1}{s_1a_1}$ is known

$$\frac{s_1t_1}{s_1a_1} = \frac{\tan i_1}{\tan r_1}$$

"(3) $\frac{s_1t_1}{s_1a_1} = \frac{t_1p_1}{a_1q_1} = \frac{\tan i_1}{\tan r_1}$ or (for print 1)

$$\frac{\text{apparent parallax value}}{\text{true parallax value}} = \frac{\tan i_1}{\tan r_1}$$

Similarily, for print 2, (Figure VI C-54):

$$\frac{\text{Apparent parallax value}}{\text{true parallax value}} = \frac{\tan i_2}{\tan r_2}$$

(4) For practical purposes;
$C_1R_1 = C_1R_2$, $R_1C_2 = R_2C_2$

(5) Then, $\frac{a_1 q_1}{a_2 q_2} = \frac{C_1R_1}{C_2R_2}$

(6) Ratio $\frac{C_1R_1}{C_2R_2}$ can be determined by dropping a perpendicular from "a" to flight line $C_1C_2$.

(7) Knowing sum $a_1q_1 + a_2q_2$ (parallax difference "s" and "a") the values $a_1q_1$ can be calculated from proportion $\frac{a_1q_1}{a_2q_2} = \frac{C_1R_1}{C_2R_2}$

(8) For print 1 ; $\frac{t_1p_1}{a_1q_1} = \frac{\tan i_1}{\tan r_1}$ , $t_1p_1 = a_1q_1 \frac{\tan i_1}{\tan r_1}$

Similarily for print 2; $t_2p_2 = a_2q_2 \frac{\tan i_2}{\tan r_2}$

(9) Then; $t_1p_1 + t_2p_2$ = true parallax value for depth hT.

EXAMPLE: KNOWN VALUES

1. Parallax difference between surface point and bottom point measures .09 mm.
2. Focal length = 8.25".
3. Distance from photo center to depth image, Photo 1 = 1 1/4", Photo 2 = 2 1/4".
4. From graph: $\frac{\tan i_1}{\tan r_1} = 1.363$, $\frac{\tan i_2}{\tan r_2} = 1.347$
5. $C_1R_1 = 1.94$, $C_2R_2 = .69$"

CALCULATIONS

1. $\frac{\text{Parallax diff.}}{C_1R_1 - C_2R_2} = \frac{a_1q_1}{C_1R_1}$, $\frac{.09}{2.63} = \frac{a_1q_1}{1.94}$ $a_1q_1 = .066$

"2. $\frac{\text{parallax diff.}}{C_2R_2 - C_2R_2} = \frac{a_2q_2}{C_2R_2}$, $\frac{.09}{2.63} = \frac{a_2q_2}{.69}$ $a_2q_2 = .024$

3. $t_1p_1 = a_1q_1 \frac{\tan i_1}{\tan r_1} = .066 \times 1.363 = .09$

4. $t_2p_2 = a_2q_2 \frac{\tan i_2}{\tan r_2} = .024 \times 1.347 = .032$

5. True parallax value = $t_1p_1 - t_2p_2 = .122$ mm

(4) Present State of Development:

Results with Fairchild Stereocomparagraph

Several photographic flights were made over Cat Cay, Bahama Islands, where 20 lead line soundings of depths between 2' and 15' had been made previously. Buoys floating on the water surface marked the points of known depth.

The results are tabulated as follows:

| Test No. | Processing | RF | H | F | $\frac{dh}{dp}$ | Average Error | Max. Error |
|---|---|---|---|---|---|---|---|
| 1 | 3 x enlargement | 1/1450 | 3000' | $8\frac{1}{4}$" | 7.4 | .9' | 3.2' |
| 2 | " | " | " | " | 14.8 | .8' | 1.8' |
| 3 | contact print | 1/4360 | " | " | 23.0 | 1.0' | 3.0' |
| 4 | " | " | " | " | 44.4 | .9' | 1.9' |
| 5 | " | " | " | " | 45.3 | 1.7' | 5.0' |
| 6 | " | 1/12000 | 6000' | 6" | 72.9 | 1.2' | 4.2' |

Average individual values for each point on the 6 tests the average error is .5', maximum error 1.6'.

CONCLUSIONS:

These trials indicate that depths can be determined with the Stereocomparagraph on a single stereoscopic pair of photos with an average error of from - 1.0 to 1.5 feet and a maximum error of - 3 to 4 feet provided the following conditions exist:

"(i) Sufficient control on surface of water to eliminate error in parallax measurements due to tip and tilt of photos. (Elimination of tip and tilt errors must be overcome before any real accuracy can be attained with this method).

(ii) Stereocomparagraph operator who has had at least 100 hours experience in making depth determination readings.

(iii) Altitude of plane known within 10%.

(iv) Well defined bottom detail on photographs.

(v) $\frac{dh}{dp}$ value of 10' to 70' per mm. on contact prints or enlargements up to 300% from original negatives.

If values of several photographic runs are averaged, a greater accuracy can be attained than when using a single pair of photos.

A too optimistic view should not be taken from the above results because the buoys on the water surface provided ideal surface control (see conditions 1 above). Ordinarily the effects of tip and tilt would seriously effect the results unless the photos could be corrected for such distortion. Usually the accuracy will be somewhat less than the above results would indicate depending on the amount of tip and tilt correction necessary. Photos taken during ideal flying conditions will closely approximate the accuracy obtained in these tests. Badly tilted photos will be entirely unusable.

The photographic conditions imposed for suitable accuracy would probably not be tactically practical in most cases.

RESULTS WITH MULTIPLEX EQUIPMENT

Eight depths in the Cat Cay area were measured with Multiplex equipment by the U. S. Geological Survey for the Photographic Interpretation Center. The flight altitude = 3000' the focal length = $8\frac{1}{4}$". The maximum error was 3.3' and the average error was 1.3'. These results, while not conclusive, would indicate that the Multiplex and Stereocomparagraph attain about the same degree of accuracy. However, the Multiplex is affected much less by the tip and tilt than is the Stereocomparagraph. Where a choice of the two devices is offered, the Multiplex is recommended.

It should be noted that the conditions of photography are the same for the Multiplex as for the Stereocomparagraph and would ordinarily be impractical tactically.

"(5) Photographic Requirements:

(a) Choice of Cameras:

"A study of the parallax equation will reveal what conditons must prevail for accurate depth determinations to be made. Obviously dh/dp must be small. That is, one unit of parallax measured on the photo must be equal to a relatively small number of feet difference in elevation. Readings may be made to about .03 mm of parallax with the Fairchild Stereocomparagraph, giving a difference in parallax with a maximum of .06 mm (plus or minus). If we wish to determine differences in elevation with a maximum of error of 1 foot (plus or minus), then dh/dp must be not greater than 16.7 ft/mm.

In water depth determination, the amount of parallax to be measured is reduced by a factor of approximately 1.4 due to refraction, and therefore dh/dp must be reduced by the same factor to give the same accuracy, or dh/dp must be not greater than 12 ft/mm.

If an accuracy of 2' is desired, the factor for 1' accuracy is simply multiplied by 2; or: dh/dp 24 ft/mm, etc. In other words, dh/dp is inversely proportional to the accuracy desired; and conversely the maximum error will be directly proportional to dh/dp. Considering the equation dh/dp = H/b, it is seen that dh/dp (and thus the maximum error) will be directly proportional to H, and inversely proportional to b. Since the length b is a function of overlap and film size in direction of flight, if the film size and overlap are constant, the only way the maximum error can be decreased is by decreasing altitude H. Any percentage change in altitude will produce approximately the same percentage change in dh/dp. To limit the maximum error to one foot (plus or minus) with 9" x 9" film and 60% overlap it is necessary that the altitude be about 1100' (solve for H where $\frac{dh}{dp} = 12 = \frac{H}{b \text{ (in mm)}} = \frac{H}{914}$).

As indicated above, dh/dp may also be decreased by increasing b; and the percentage change in each will be approximately equal. The photographic flight base b may be increased by decreasing overlap or by increasing film size or by both.

It is not necessary to have 50% or more overlap in order to make depth determination as long as the flight base b can be determined and sufficient relief control is present in the area of overlap. Since a decrease in overlap will give a larger b, the same dh/dp and consequently the same accuracy can be attained at a greater altitude than if the standard 60% overlap is used. For example, 20% overlap on 9" x 9" film will give an accuracy of 1' at 220', twice the altitude required for 60% overlap.

Complete stereo coverage can still be obtained with this method by setting the camera for a percentage overlap equal to 100% minus the overlap desired. For example, if we want 20% overlap, a camera setting of 100% - 20% or 80% will allow us to view the whole area stereoscopically at 20% overlap by using successive pairs of photos 4 numbers apart in

"the run (1 and 5, 2 and 6, etc.).

"To increase b by increasing film size, it is necessary either to use a camera with a larger film size in the direction of flight, or to make enlargements of the contact prints or both. Using present equipment in the standard manner, the maximum film dimension in the direction of flight is 9".

The K-18 camera, which has a 9" x 18" film with the 9" in the direction of flight, could be rotated 90° in its mount to place the 18" film dimension in the direction of flight. Thus with 20% overlap, the maximum error could be limited to 1 foot at an altitude of 4400', and the same accuracy could be attained at 2200' with 60% overlap. A sufficiently clear photograph cannot be made at this altitude with the K-18 camera.

The effect of making enlargements of the contact prints is to increase b by the amount of enlargement, i.e. if the contact print is doubled, b will be doubled. The limiting factor here will depend on how many times the contact prints can be enlarged without losing significant detail. It is thought that an enlargement of at least 2 or 3 times will be possible if sufficient care is taken in the processing of the film and enlargements. By using this method, then, it should be possible to attain the same accuracy as given in the previous examples at two to three times the altitudes given, with other conditions remaining the same.

Certain mechanical and optical factors other than film size place limitations on the altitude and flight base (and thus the dh/dp) obtainable in any specific case. These include shutter speed, minimum time for camera rewind, and speed of the plane. If the recycle time is longer than needed for the desired overlap, it should be possible to double the overlap by using a pair of cameras, taking successive photos on alternate cameras. The more important limiting factors for the F-56, K-18, K-24, and K-25 cameras are shown in the following tables:

## Camera Characteristics

TABLE VI C-IV

| Camera | Focal Length, inches | Maximum Shutter Speed, seconds | Film size, inches | Recycle Time, seconds | Minimum Altitude of Operation at 250 knots, feet * | Maximum overlap at this altitude, percentage | dh/dp at this altitude using 60% overlap, contact print, ft/mm** | dh/dp at this altitude using 20% overlap 3 diameter enlargements** | Maximum altitude 2 foot max error, contact print 60% overlap, feet | Maximum altitude 2 foot max. error, 3 diameter enlargement, 20% overlap, feet |
|---|---|---|---|---|---|---|---|---|---|---|
| F-56 | 5¼ | 1/150 | 7x7 | 0.8 | 2900 | 91 | 41 | 7 | # | 10,200 |
| | 8¼ | 1/225 | 7x7 | 0.8 | 3100 | 87 | 44 | 7 | # | 10,200 |
| | 20 | 1/225 | 7x7 | 0.8 | 7500 | 87 | 105 | 17 | # | 10,200 |
| | 40 | 1/150 | 7x7 | 0.8 | 22,500 | 91 | 316 | 52 | # | # |
| K-17 | 6 | 1/300 | 9x9 | 2.4 | 1700 | 60 | 19 | # | 2200 | 13,200 |
| | 12 | 1/225 | 9x9 | 2.4 | 4500 | 70 | 49 | # | # | 13,200 |
| | 24 | 1/150 | 9x9 | 2.4 | 13,400 | 80 | 147 | 25 | # | 13,200 |
| K-18*** | 24 | 1/150 | 9x18 | 3.0 | 13,400 | 87 | 73 | 12 | # | 26,400 |
| K-24 | 6 3/8 | 1/900 | 5x5 | .33 | 620 | 60 | 13 | 2 | 1200 | 7,200 |
| K-25 | 6 3/8 | 1/500 | 4x5 | .5 | 1,200 | 82 | 24 | 4 | 1200 | 7,200 |

* Assuming circle of confusion of 1/200 inches and f 11 stop.

** To figure probable maximum error in feet divide dh/dp by 12.

*** 18" film dimension is in direction of flight.

Author's note: This table should be brought up to date.

"A study of this table reveals that, if the conditions of operation specified above (which are thought to be typical) are met, the choice of cameras is considerably limited for accuracy required.

Is should be noted that the accuracy of depth determination is not only dependent on $\frac{dh}{dp}$ value but also on the accuracy of altitude. Above 1000' feet, errors in altitude usually have slight effect. However, at lower altitudes, the percentage error in altitude may become relatively large. This error can be held at a minimum by using a radio altimeter. Even with this special altimeter, as much as 10% error in depth might be expected in addition to the errors involved in photogrammetric measurement.

For very low altitudes (200-600') the K-24 camera is the only camera available that will provide suitable overlap at these altitudes. The errors involved might cause a maximum error in depth greater than 20%. At plane speeds of 250 knots, the image will be slightly blurred.

For low altitudes (600-1500') the K-25 camera would be most practical at these altitudes. A depth error greater than 10% might be expected. At plane speeds of 250 knots the image will be slightly blurred when the altitudes are less than 1200 feet.

For medium altitudes (1500-12000') the K-17 camera with 6" lens would provide best results at these altitudes. An error greater than 1.5' to 2' in depth might be expected. At plane speeds of 250 knots the image will be slightly blurred when altitudes are less than 1700 feet.

For high altitudes (above 12000') the K-18 camera with 18" dimension in line of flight provides best results at high altitude. An error greater than 2' might be expected at 12,000', with accuracy decreasing with altitude. At plane speeds of 250 knots, the image will be slightly blurred at altitudes less than 13,500 feet.

It should be emphasized that the errors listed are for theoretically perfect pictures. Additional errors will be induced by tip and tilt; the amount depending on the flying conditions.

(b) Exposure and Processing:

Films and Filter: Good surface and bottom detail may be obtained with either black and white or Aero Koda-color films. For best bottom detail which is necessary with this method, no filter is used on the camera lens for the black and white films. If haze conditions are met, it is best to use the minimum haze correction filters required to penetrate the haze. Good results have been obtained with films such as Special Aero Pan, and Super XX. Aero-Infra-Red film, also used with no filter, gives very good water penetration due to its high blue speed.

Lighting: Sunlight is necessary to show good surface and bottom detail. The best sun angle is in the range of 30 to 60 degrees.

"Exposure: With the black and white films, exposure should be on the full side in order to obtain good bottom detail. In bright sun, this detail has been obtained on Special Aero Pan, using no filter, at 1/150 second at f11; and on Aero-Infra-Red with no filter at 1/150 second at f5.6 to f8. For Aero Kodacolor, (using the filters recommended by the Eastman Kodak Company for this altitude), best results have been obtained at 1/150 second at f6.3. With Aero Kodacolor, it is advisable to open up the aperture one-half stop, to f8, for bright bottom. These exposure recommendations are for January, latitude approximately 25° north.

Development for average to above average contrast is recommended. With Special Aero Pan, 20 minutes at 68° F, and with Super XX Aero Pan, 25 minutes at 68° F, in DK-50 on the Smith Aerial Film Developer, have given satisfactory results. This gives a contrast (gamma) of approximately 1.4. Aero-Infra-Red exposed without a filter requires only 8 minutes in DK-50, or, better, 10 minutes in D-76.

Prints should have as high a contrast as is possible and still keep the highlight (surface) and shadow (bottom) detail. With a dark bottom a compromise of lower contrast is necessary. If Aero Kodacolor is used, it may be desirable to make a duplicate negative, and a positive print by contact. In order to have correct orientation, it is necessary to make the print with the base of the dupe negative film in contact with the paper emulsion. Some loss in sharpness is caused by this method of contact printing, but usable results have been obtained. Projection printing can be used for better sharpness. Duplicate Kodacolors can also be made and used as transparencies.

(6) Working Technique:

With both the Stereocomparagraph and the Multiplex, depth measurements are similar to ordinary topographic measurements except that refraction corrections must be made. It is not the purpose of this report to describe the general operation of these instruments, but only to adapt them for depth determination.

(a) Fairchild Stereocomparagraph: The operator should familiarize himself with a manual for general operation of this instrument before attempting any measurements.

Tip and tilt should be carefully corrected. There are several methods for correcting the photos. Probably a correction graph (see working manual for Stereocomparagraph), a rotation system (see "Stereoleveler" in P.I.C. Report Underwater Depth Determination from Aerial Photographs OpNav 16-V-P-30), or a combination of the two would be most practical for depth determination. The operator should keep in mind that gross errors in parallax will result from uncorrected photos.

"Instead of running contour lines as is usually done in topographic work, spot elevations (depths in this case) should be measured. This affords greater accuracy because several readings can be taken for each depth. At least five readings should be made for each point.

The refraction corrections should be applied as described on page b(3). It should be remembered that the measurements from center of photo to the depth image point for refraction correction should be made on the original negative or a contact print.

(b) Multiplex Equipment: The multiplex is a specialized device and should be operated only by experienced personnel.

The technique for depth determination is similar to that for topographical work. However, as in the case of the Stereocomparagraph, spot depths should be measured instead of running underwater contour lines. At least five readings should be taken for each depth.

The refraction corrections should be applied exactly as when using the Stereocomparagraph."

(End of quote from Reference VI C-4 pp. 3.01-3.09)

c. Double Cameras: (Reference VI C-4, pp. 4.01-4.09)

"(1) Installation: This method of depth determination depends upon mounting two cameras in an airplane with a fixed distance of at least 20' between them. For accuracy with this method, low flying altitudes (below 300') are necessary. This necessitates the use of a fast fighter-type plane in combat areas where high speeds are necessary tactically. In order to maintain a camera base greater than 20' in a fighter plane, the cameras must be installed in the wings.

The two cameras photograph the water surface simultaneously, providing a stereoscopically visible surface datum from which to measure depths.

(2) Parallax: The theoretical parallax relationships are similar to those of other stereoscopic depth determination methods. However, by having a fixed distance between cameras, the parallax formulae are not dependent on the altitude. This is an advantage because all altitude instruments, even the radio Altimeter, are subject to error.

(3) Derivation of Parallax Formulae(Figure VI C-56)

Symbols for Figure VI C-56

$C_1$ = Location of Right Camera
$C_2$ = Location of Left Camera
B = Distance between Cameras
H = Plane altitude

"L = Apparent Depth
O = Surface Image Point
Q = Bottom Image Point
F = Camera Focal Length

When bottom image point "Q" establishes "b" (see section on Working Techniques 4(c)(7).

1. $\frac{P}{b'-p} = \frac{P}{B}$ 2. $\frac{h}{P} = \frac{F}{b'}$ 3. $h = \frac{PF}{b'}$

4. $P = \frac{pB}{b'-p}$ 5. $h = \frac{pBF}{b'(b'-p)}$

When surface image point ")" establishes "b" (see section on Working Technique):

1. $\frac{p}{b} = \frac{P}{B}$ 2. $\frac{h}{P} = \frac{F}{b-p}$ 3. $h = \frac{PF}{b-p}$

4. $P = \frac{Bp}{b}$ 5. $h = \frac{pBF}{b(b-p)}$

Possible Errors Involved

(i) Tip and tilt: all errors caused by tip and tilt will be small if the tip and tilt is less than 15°.

(ii) Imperfectly aligned cameras: errors due to improper alignment of the two cameras will be large unless the installation is calibrated as described later in this report. C(b)C Calibration of installation. The errors will probably become inconsequential when a calibration correction is applied.

(iii) Wing deflection: bumpy air will cause the plane wings to deflect, momentarily throwing the cameras out of alignment. Large errors are liable to result unless the deflection can be detected. Only with the strip camera can deflection be detected and corrected for. If standard cameras are used in bumpy air, large depth errors will result.

(4) Refraction Correction

When the above formulae are used for underwater depth determination, a correction must be made for refraction at the air-water interface. The refraction correction for two standard aerial cameras (such as the K24) is exactly the same as described under the Standard Aerial Camera section of this report. The refraction correction for two strip cameras is derived as follows: Figure VI C-57

## "Derivation of Refraction Correction for a Stero-Pair of Strip Photographs

Symbols for Figure VI C-57

OE = True Depth
OA = Apparent Depth, Photo 1
OD = Apparent Depth, Photo 2
OB = Apparent depth in Stereo
C = Image in Stereo
$j_1$ = Angle of Incidence, Photo 1
$j_2$ = Angle of Incidence, Photo 2
$r_1$ = Angle of Refraction, Photo 1
$r_2$ = Angle of Refraction, Photo 2

---The angle of incidence, i, is determined as follows:

$$\tan i_1 = \frac{d_1}{F}, \quad \tan i_2 = \frac{d_2}{F}$$

where $d_1$ = distance in inches from principal point of photo 1 to the image of E on photo 1.

$d_2$ = distance in inches from principal point of photo 2 to the image of E on photo 2.

F = Focal length of lens in inches.

The angle of refraction is determined by the relationship sin 1/sin r = 1.336 (for salt water).

---$OE = OA \frac{\tan i_1}{\tan r_1}$ or $OA = OE \frac{\tan r_1}{\tan i_1}$

$OE = OD \frac{\tan i_2}{\tan r_2}$ or $OD = OE \frac{\tan r_2}{\tan i_2}$

---In ABC, $\frac{BC}{AB} = \tan i_1$

In BCD, $\frac{BC}{BD} = \tan i_2$

---Dividing 3 (a) by 3 (b), $\frac{\tan i_1}{\tan i_2} = \frac{BC}{AB} \times \frac{BD}{BC} = \frac{BD}{AB}$

or $BD = AB \frac{\tan i_1}{\tan i_2}$

---$OB = OA + AB = OE \frac{\tan r_1}{\tan i_1} - AB$

$$\text{OB} = \text{OD} - \text{BD} = \text{OE}\,\frac{\tan r_2}{\tan i_2} - \text{BD}$$

---Substituting 4 in 5 (b), $\text{OB} = \text{OE}\,\frac{\tan r_2}{\tan i_2} - \text{AB}\,\frac{\tan i_1}{\tan i_1}$

---Clearing 5(a) and 6 of fractions

$$\text{OB} \tan i_1 = \text{OE} \tan r_1 - \text{AB} \tan i_1$$

$$\text{OB} \tan i_2 = \text{OE} \tan r_2 - \text{AB} \tan i_1$$

---Adding 7(a) and 7(b)

$$\text{OB} \tan i_1 + \text{OB} \tan i_2 = \text{OE} \tan r_1 + \text{OE} \tan r_2 + \text{AB} \tan i_1 - \text{AB} \tan i_1$$

$$\text{OB} (\tan i_1 - \tan i_2) = \text{OE} (\tan r_1 - \tan r_2)$$

---True depth OE = Apparent depth, $\text{OB}\,\frac{\tan i_1 - \tan i_2}{\tan r_1 - \tan r_2}$

In this case $\frac{\tan i_1 + \tan i_2}{\tan r_1 + \tan r_2}$ may be called the correction factor.

See Figure VI C-59 for correction factor graph.

(5) Present State of Development

(a) Tests at Pearl Harbor, Hawaiian Islands

In experiments carried out by the Joint Intelligence Center, Pacific Ocean Area, two K24 cameras were mounted in a PB4Y type aircraft in fore and aft positions giving a 20' camera base. Five depths ranging from 1.7' to 3.6' were measured with a maximum error of .7'. Unfortunately, these tests did not cover a sufficient number and range of depths to adequately test the installation. However, the experiment suggests that a two camera installation has good accuracy for depth measurement. The effect of mounting the cameras in the wings, rather than in the rigid plane body, and the accuracy over a large number of depths of suitable range must be ascertained in future tests before the tactical value of a two camera installation can be determined.

(b) Report by National Defense Research Committee:

Report No. 1, Soundings and Beach Contours with Airplane Cameras Mounted Athwartships, by Philip Kissam, published the National Defense Research Committee, Aug. 3, 1944, concerns the parallax formulae and calibration of the installation. This initial report was published as an aid and stimulus to further experimentation with a two camera installation. While incomplete, this publication is useful for an understanding of the theoretical parallax relationships.

*(c) Tests at Cat Cay, Bahamas:

At the request of the Photo Interpretation Center, VD-2 at Norfolk, Va. is making a two camera installation in an F6F type plane.

Because it is felt that the single lens strip camera has marked photogrammetric advantage over the K24, this installation is being made for two strip cameras. However, if desired, the installation can be easily adapted to accommodate two K-24 cameras.

At the present time a mock-up has been made which demonstrates the practicability of the installation. The wind structure will not be seriously altered and the flying characteristics of the plane will not be appreciably changed. The camera base is 27'.

It is expected that the installation will be tested over an area of known depths ranging from 2' to 15' at Cat Cay, Bahama Islands.

(d) Conclusions:

While the bomber installation at Pearl Harbor proved of value in the early experiments, it is felt that a fighter plane installation is necessary in combat areas.

From theoretical considerations and the Pearl Harbor experiments, it is considered possible that a two camera installation might have somewhat better accuracy than the Sonne Stereo-strip method of depth determination.

(6) Photographic Requirements

(a) Photographic Conditions

The photographic conditions are similar to those described for the Sonne Stereo-strip method of depth determination.

(b) Choice of Cameras

Because of the low altitude and high plane speed necessary photogrammetrically and tactically, either a camera with a very fast shutter speed or a strip camera is necessary.

The K24 is the only standard aerial camera that can photograph clearly with suitable stereoscopic overlap (20%) under the conditions imposed.

The K24 camera has the following characteristics:

Focal length = 6 3/8", 7", 12", and 20"
Shutter speed = 1/900 second
Recycle time = 1/3 second
For depth determination the 6 3/8" or 7" lens should be used.

"The Sonne single lens strip camera (not to be confused with the Sonne double lens stereo-strip camera) is capable of photographing at any altitude below 1000' at extremely high plane speeds. The focal length is 6".

In order to obtain the high shutter speeds in the K24, a focal plane shutter is necessary. This type shutter causes certain photogrammetric errors which increase with plane speed. Because of the nature of the installation, these errors will probably be small. Wing deflection, which might cause serious errors, cannot be detected with K24 cameras. In view of the above possibilities of error when using K24 cameras, plus the fact that the Sonne camera has a much larger field of view, it is recommended that the Sonne single strip cameras be used in the camera installation.

(c) Calibration of Installation:

Because it would be extremely difficult to mount the cameras in perfect alignment it is suggested that each installation be calibrated.

A circular target of a diameter equal to the camera base (distance between cameras) should be marked out on the ground. Photographs should be taken of the calibrating target in the same manner as in operational use of the installation. If the cameras are perfectly aligned, the photo distance between camera centers (the distance on the left photo between its center and the image that falls at the center of the right photo) will be exactly equal to the photo diameter of the target. The degree that the cameras are out of alignment can be determined by measuring the difference in photo distance between camera centers and the target diameter. This is called the calibration distance.

The calibration distance is plus if the photo center distance is smaller than the calibration target's photo diameter, and minus if the photo center distance is larger.

Whenever calculating the term "b" in the parallax formulae (the photo distance between camera centers) the value of the calibration distance is added or substracted, according to its sign, to the measured "b" distance. This correction is necessary every time the parallax formulae are used.

(7) Working Techniques:

(a) Using Two Sonne Single-Strip Cameras

Preparation of Photographs: The center line should be located and plotted on each photographic strip. The two strips should then be oriented side by side so that the center line of one strip is parallel to the center line of the other. In this position the photos are ready for parallax measurements.

"Parallax Readings: There are two methods of reading the parallax value between the stereoscopic surface of the water and the bottom:

(i) By mounting a parallax bar perpendicular to the center lines of the strip photo and reading the parallax stereoscopically.

(ii) By reading, with a fine scale, distance between the surface image and the bottom image on each strip. The difference between the two photo distances of the surface image and bottom image measurements is the parallax value between the surface and bottom. Measurement is always perpendicular to flight lines (center lines).

The measurements with the parallax bar will be more accurate, although it requires practice to operate the instrument properly.

The method of making the parallax readings stereoscopically (with a "floating dot" instrument such as stereocomparagraph or parallax bar) is described in the Sonne Stereo-Strip and the Standard Aerial Camera methods of depth determination.

Parallax Formulae: In order to use the parallax formulae, the photo distance between camera centers (term "b" must be obtained. Figure VI C-60. This is done by plotting the left strip, a line connecting the images falling upon the center line of the right strip. It will be noticed that the surface image points will plot on a slightly different line from the bottom image points. This is due to parallax difference. Either set of image points can be plotted, but they should be consistent with the parallax formula used. The distance "b" is the distance between the center line of the left photo and the plot of the center line of the right photo (see Figure VI C-60). The value of "b" should be corrected according to the installation calibration.

If the surface center points are plotted, the formula used is

$$h = \frac{pBF}{b(b-p)}$$

If the bottom center points are plotted, the formula used is

$$h = \frac{pBF}{b(b-p)}$$

where,

h = Apparent vertical distance from surface to bottom
B = Fixed base distance between cameras
F = Focal length of cameras
b = Photo distance of B (distance between film centers) corrected by calibration distances
p = Parallax measured between surface and bottom

The terms "h" and "B" should be measured in the same units (usually millimeters or thousandths of feet).

"The formulae used for depth determination now become;

(a) Where surface image established "b":

$$\text{Depth} = \frac{pBF}{b(b-p)}$$

Example

$p = .3$ mm $\qquad F = 152$ mm $\qquad n = 1.41$

$B = 27'$ $\qquad b = 20$ mm

$$\text{Depth} = \frac{.3 \times 27 \cdot 152}{19.7(19.7-.3)} \times 1.41 = 4.4'$$

Wing Deflection: Air currents cause bumps which deflect the plane wings. This throws the cameras out of alignment and will cause large errors in the measurement of "b". This in turn will affect the formulae and the final result. The plot of the center lines will readily show where deflection has occurred (see Figure VI C-60). To compensate for deflection, the distance "b" is obtained where the flight is obviously undisturbed. Deflection will not affect parallax appreciably, therefore the use of an undisturbed or average "b" value is the only correction necessary for wind deflection.

(b) Using Two K24 Aerial Cameras:

Preparation of Photos: The two photographs are oriented with the flight lines parallel. Otherwise the photographs are prepared exactly as described for the Standard Aerial Camera method.

It has been found that the accuracy of parallax measurement can be increased by enlarging the photographs up to 3 diameters.

Parallax Formulae: The parallax formulae for the two camera installation applies to the K24 camera exactly as to the Sonne single strip camera. The term "b" must be corrected by the calibration distance.

The true depth formulae are:

(i) Where surface image establishes "b",

$$\text{Depth} = \frac{pBF}{b(b-p)} \times n$$

(ii) Where bottom image establishes "b",

$$\text{Depth} = \frac{pBF}{b(b-p)} \times n$$

("n" is found as described under section on Standard Aerial Cameras.)

The parallax measurements, and the refraction corrections are accomplished exactly as described under the method of depth determination with Standard Aerial Cameras. However it should be remembered that the parallax formulae are different."

(End of quote from Reference VI C-4 pp. 4.01-4.09)

5. Appendix

a. References Cited

VI C-1. The Committee on Amphibious Operations (Notes by), National Research Committee, March 29, 1951. Unpublished. CONFIDENTIAL

VI C-2. Crooke, R.C., R.L. Wiegel, O.J. Koester (Captain USMC), and G.E. Thomas (M/sgt USMC), Beach Profile Determination from Timed Vertical Aerial Photographs, Technical Report 155-48, Office of Naval Research, Contract N7onr-29519, University of California, Berkeley, June, 1951.

VI C-3. Johnson, J.W., Progress Report: Wave-Velocity Method of Depth Determination by Aerial Photographs, Technical Report No. 155-10, Office of Naval Research, Contract N7onr-29519, University of California, Berkeley, October, 1949.

VI C-4. Photographic Interpretation Center Report #46, Underwater Depth Determination, OPNAV 16- VP 46, October, 1944. CONFIDENTIAL

VI C-5. Proof Test of Water Transparency Method of Depth Determination, Beach Erosion Board, Corps of Engineers, Engineer Intelligence Division Office, Chief of Engineers, Department of the Army, July, 1948.

VI C-6. R.A.A.F. Report Detail No. 814/A/261 - ONI M-1822-46, The Transparency Method of Depth Determination, May, 1946.

VI C-7. Williams, W.W., "The Determination of Gradients on Enemy-Held Beaches", The Geographical Journal, June, 1946, pp. 76-93.

5. Appendix

b. Methods of Presentation (Reference VI C-4 pp. 8.01-8.02)

"The results of depth determination work will seldom, if ever, constitute a separate report, but will form a section or part of a more comprehensive study. For this reason no specific methods of presentation can be set forth; the information will have to be fitted into its proper place in the larger document. There are, however, some problems of presentation that may be considered here.

Hydrographic information, of the sort provided by photographic methods, will have its entire application in amphibious operations.

The work of an amphibious group for any given operation may be roughly divided in two parts (1) Planning Phase, and (2) Operational Phase. Information for each phase will require its own method of presentation.

(1) Planning Phase:

During this period the material will have to be shown to only a small group of people, and it will generally not have to be presented in too great detail. Graphic methods of presentation seem to be best, although some supplementary written information may be necessary.

A map or chart of the target area should be kept, with the beaches under consideration marked on it. On this chart all section lines may be plotted, and the underwater gradient written directly on the section line. All reefs, sandbars, or other major obstructions found off any of the beaches should be plotted and depths over them indicated where possible. The suitability of each beach for types of landing craft should be noted on the chart. The table of beaching characteristics of landing craft, Table 5 Appendix e and Figure VI C-64, will be helpful in this respect. Profiles of the bottom along each section line should be plotted on separate sheets of graph paper and if enough points can be determined bottom contours may be sketched in on a large scale chart or an overlay made from the photographs. Overlays should be gridded with the standard military grid that will be used in the operation.

All material, whether completed or not, should be kept in presentable form at all times. The interpreter will often have to explain his work to others.

(2) Operational Phase:

For distribution to the personnel participating in the assault, a different problem of presentation arises. Here the information must be given, in easily understandable form, to large groups and in the greatest possible detail. Each amphibious force will have its own methods of presenting the Intelligence Annex to the operation plan, and hydrographic information must conform to the general pattern. However, other graphic documents are often

issued to landing craft skippers, beach parties, scouts and raiders, and other special groups. Whether these graphic documents be annotated charts, maps, or simply overlays from photographs, or sketches, they should contain as much detailed hydrographic information as is available. Fathom lines should be marked, high and low water lines indicated, all underwater obstructions accurately positioned with their depths written alongside, where known. The gradient of the beach at various points along its length should be included, and any deep water channels that may help craft get in closer to shore should be noted. The magnitude and direction of currents along shore should be indicated.

Wherever possible maintain the same scale for various graphic presentations for one operation. Make an effort to maintain a simple scale relationship between charts and drawings; for example, if charts are at a scale of 1:50000, a convenient scale for drawings might be 1:25000 or 1:12500.

Use standard symbols where possible; either those standard with the group with whom you are working or those published by the Hydrographic Office (Bulletin #8).

Keep your presentation as simple as possible. If necessary use overlays to indicate beach and underwater obstructions; it is better than crowding a sheet so that it is not understandable. Use transparent colors, but only when they serve to clarify and simplify the presentation. Two shades of light blue will aid in distinguishing the beach, the area exposed between tides, and the area below the low water line.

These are only the general considerations that should always be remembered; special cases will require additional information.

All that need be emphasized is the fact that the hydrographic information resulting from photographic determinations forms only a small part of the beach intelligence necessary to an operation, and that it must be coordinated with other elements, and be presented as a part of the whole picture. Graphic methods of presentation should be used where possible, since they are easy to understand, and the hydrographic data can be better related to other information if both are given a graphic form.

The following enclosures represent two methods of presentation which have been used for European operations, Figures VI C-61, 62, 63. Neither is considered a standard, though, that for "Utah" beach, is extremely compact and complete. However, neither shows underwater data in as complete a form as is necessary or desirable."

"The Hydrographic Office will publish in the future a report, tentatively titled 'Amphibious Sketches'. The purpose of this report will be to provide a standard form for the graphic presentation of beach and hydrographic intelligence."

(End of quote from Reference VI C-4 pp. 8.01-8.02)

## 5. Appendix

### d. USEFUL FACTORS AND TABLES

1. Mph to knots: Mph x .868 = knots. $\frac{\text{mph}}{\text{knots}} = \frac{38}{33}$
   Knots to mph: Knots x 1.152 = mph.

2. Mph to feet per sec: Mph x 1.466=ft.per.sec.
   $\frac{\text{ft.per.sec.}}{\text{mph}} = \frac{44}{30}$

3. Knots to ft.per.sec: Knots x 1.69=ft.per.sec.
   $\frac{\text{ft.per.sec.}}{\text{Knots}} = \frac{93}{55}$

4. Inches to millimeters: Inches x 25.4 = mm
   $\frac{\text{inches}}{\text{mm}} = \frac{26}{660}$

   Millimeters to inches: Millimeters x .0394 = inches.

5. Feet to millimeters: Feet x 305 = mm
   $\frac{\text{ft}}{\text{mm}} = \frac{1}{305}$

   Millimeters to feet: Millimeters x .00328 = ft

Thousandths of feet to millimeters:
Thousandths x .305 = mm
$\frac{\text{Thousandths}}{\text{mm}} = \frac{200}{61}$

Millimeters to thousandths of feet:
Millimeters x 3.28 = thousandths

6. Degrees of Longitude to time:
   Degree x 4 = minutes of time.
   Time to degrees of Longitude:
   Minutes of time x .25 = degrees.
   $\frac{\text{minutes of time}}{\text{degrees longitude}} = \frac{4}{1}$

7. When using the velocity formula with the Sonne Camera it is necessary to correct the plane's indicated air speed to true air speed. The table below gives correction factors at various air temperatures when the altitude is less than 400'. This table does not apply for altitudes greater than 400'.

| Temperature (Centigrade) | A/S Factor |
|---|---|
| -30° C | .916 |
| -20° C | .938 |
| -10° C | .956 |
| 0° C | .970 |
| 10° C | .989 |
| 20° C | 1.006 |
| 30° C | 1.025 |
| 40° C | 1.043 |
| 50° C | 1.058 |

Example; Altitude = 200'
Temperature = 30° C
Indicated Airspeed = 350 mph
Indicated air speed x factor = true air speed.
350 x 1.025 = 359 mph

(From Photographic Interpretation Center Report #47 Underwater Depth Determination, 1944 See Reference VI C-4)

5. Appendix

e. BEACHING CHARACTERISTICS OF LANDING CRAFT

The following table of characteristics is simplified to include only those aspects of craft critical to beaching. These are the characteristics that determine whether or not a craft is suitable for use on a given beach. Only major operational types of craft are listed.

| TYPE OF CRAFT | FULL DESIGNATION | LENGTH OF CRAFT | BEACHING DRAFT | | BOTTOM SLOPE OF CRAFT |
|---|---|---|---|---|---|
| | | | FORWARD | AFT | |
| LCA | Landing Craft (Assault) | 41' - 6" | 1' - 9" | 2' - 3" | 1:51 |
| LCVP | Landing Craft (Vehicle, Personnel) | 36' - 0" | 2' - 2" | 3' - 0" | 1:40 |
| LCP(R) | Landing Craft Personnel(ramp) | 36' - 8" | ------- | 3' - 6" | 1:100 |
| LCP(L) | Landing Craft Personnel(large) | 36' - 8" | ------- | 3' - 6" | 1:100 |
| LCS(S)2 | Landing Craft Support (small) mark 2 | 36' - 8" | 3' - 6"(max) | | Capable of beaching |
| LCS(L)3 | Landing Craft Support (large) mark 3 | 158' - 5" | 4' - 6" | 5' -10" | 1:120 |
| LCM(3) | Landing Craft Mechanized (mark 3) | 50' - 0" | 3' - 0" | 4' - 0" | 1:50 |
| LCC(2) | Landing Craft Control(mark 2) | 56' - 0" | 3' -11"(max) | | Not intended to beach |
| PGM | Motor Gunboat | 110' - 0" | 6' - 9"(max) | | Not intended to beach |
| LCT(5) | Landing Craft Tank(mark 5) | 117' - 6" | 2' -10" | 4' - 2" | 1:105 |
| LCT (6) | Landing Craft Tank(mark 6) | 120' - 5" | 3' - 4" | 4' - 0" | 1:200 |
| LCI(L) | Landing Craft Infantry (large) Nos. 1 to 350 | 158' - 5" | 2' - 8" | 4' -10" | 1:71 |
| LCI(L) | New Model Nos. 350 to | 158' - 5" | 3' - 0" | 5' - 0" | 1:79 |
| LSM | Landing Ship Medium | 203' - 6" | 3' - 5" | 6' -11¼" | 1:58 |
| *LST | Landing Ship Tank | 328' - 0" | 3' - 1" | 9' - 6" | 1:50 |
| APD | Converted "Flushdeck"Destroyers for landing raiding parties | 314' - 4" | 11' -11"(max) 4' sound dome | | Not intended to beach |

* All new models are rigged to carry 2 sections of standard pontoon causeway. Each causeway s section is 175 feet long; light draft 18 inches. Older model LST's can carry same equipment but special rigging is required.

Note on Anchorage Depths: All LCTs, LCIs, LSTs, and LSMs carry 150 fathoms of anchor cable or chain. Maximum good anchorage for these is 50 fathoms. Ground tackle on craft smaller than LCT is not standardized.

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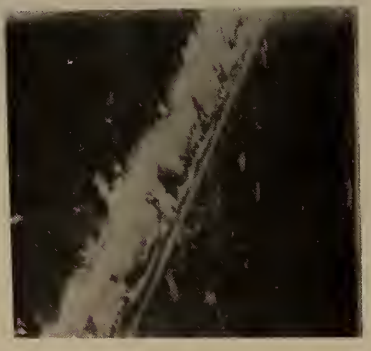


FIG. XI - BEACH TYPE A. LONG EXPOSED BEACH
The length of beach is equal to many hundreds of wave lengths. River outlet may pass through the beach at some point. On a straight beach there is more than likely one or more intermittent offshore bars. Note irregularity of surf due to the presence of bars.

BEACH TYPE B. LARGE EXPOSED BAY
This type beach will usually terminate at rocky headlands at both ends. Beaches and surf are regular because refraction reduces variation in direction at breaker line. Beaches in the central portion can approach beach type A. In the pictures at right note the lack of surf action at and near the jetties. This is due to due to wave refraction over a submarine canyon which terminates just off the ends of the jetties.

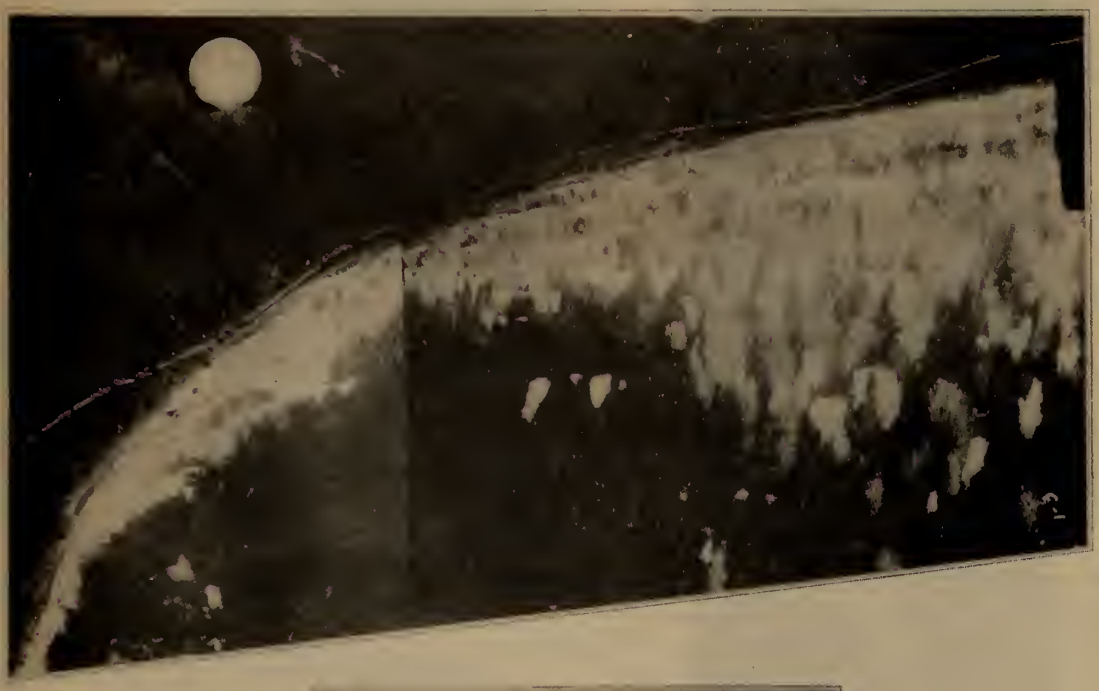




FIG. VIC-2 - BEACH TYPE C: HOOKED BAY

Note increased wave action from left to right. This is caused by by protection from the point, and refraction. The offshore rocks and reefs which project from the point are very characteristic of this type of beach. The right hand end of the beach may or may not become a type A (long, straight) beach.






FIG. VIC-3 - BEACH TYPE C: HOOKED BAY







FIG. VIC-4 - BEACH TYPE D: POCKET BEACH
Total length of beach is equal to a few wave lengths. Beaches are relatively narrow and gently curving. Profile depends on supply of beach material and exposure to wave action. Ends of beach usually terminate at cliffs or very narrow beaches at the headlands. Note the presence of a small stream which crosses the beach. This is very typical, but its location on the beach will vary from one beach to another.

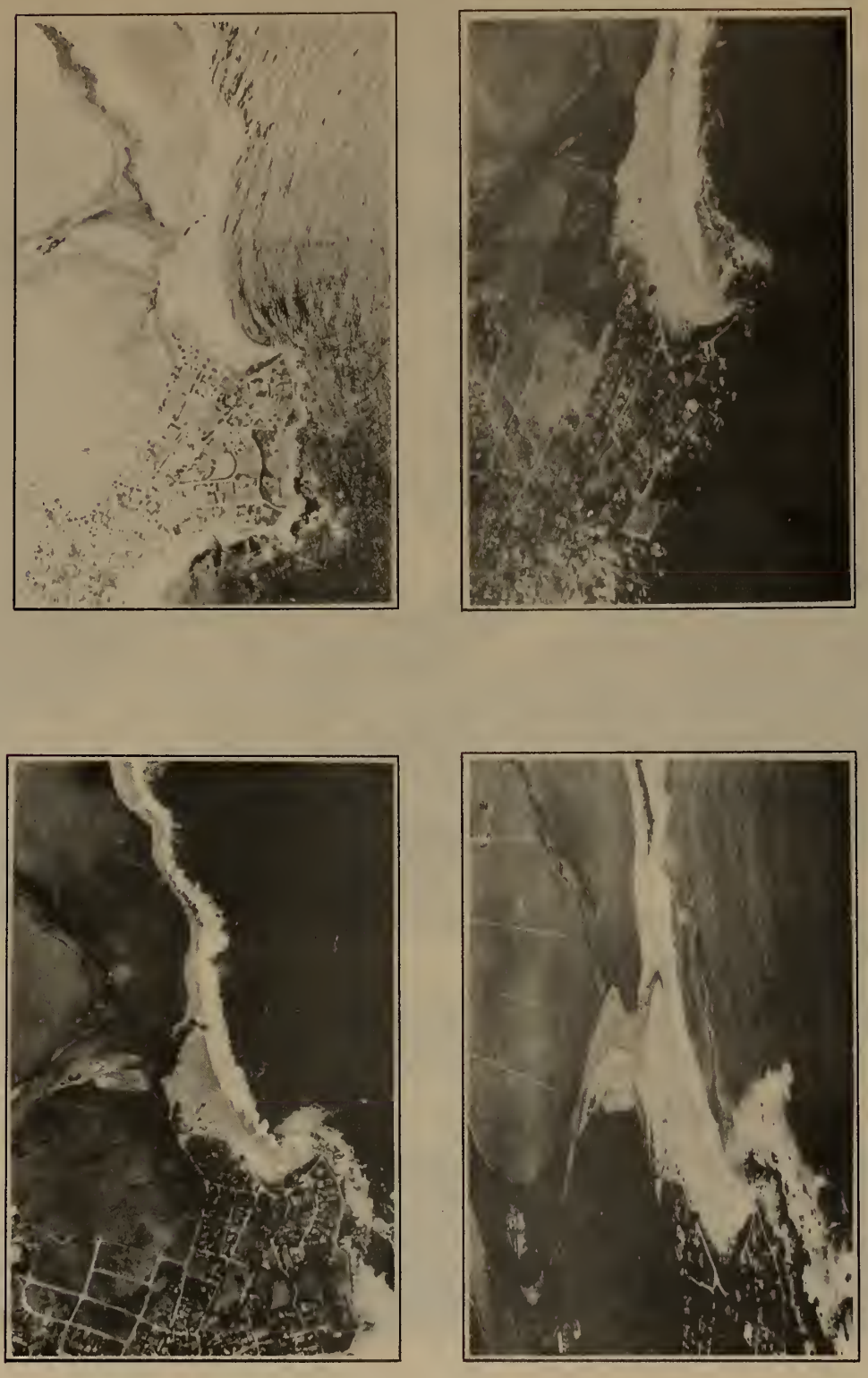


FIG. XIC-5 - Changes at o river mouth. The pictures ot the left were token in the winter, when the river dischorge is lorge. The pictures at the right were taken during the dry summer manths when the mouth is closed. This beach is classified os o pocket beoch, but it should be noted that it olso has the characteristics of o hooked boy beoch.





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(SIGNATURE) Mark Brown (RANK) Civ

(a) High surf. Note presence of bars shown by the breaking and reforming of waves. (High tide)

(b) Medium surf. Note the inner bar is partially uncovered at this tide stage. By noting the tide stage some information can be gained as to the size of the bars.

(c) Light surf, during low tide. Note the exposure of the inner two bars and channels at this low tide stage. Presence of outer bar is indicated by breaking waves. Inside of the outer bar very short wave lengths are present, indicating a breaking of the primary waves into secondary waves.

SURF CONDITIONS ON A FLAT BEACH
(Slope 1:100)
Note differences in scale of the pictures

SURF CONDITIONS ON A STEEP BEACH
(Slope 1:15)
Note differences in scale of the pictures

FIG. V-6



FIG. VIC-7 - An excellent photograph for showing the predominate direction of littoral current: In this picture the current is from left to right as shown by the fill on the left side of the jetty and by the erosion on the right side of the channel. An oblique photograph is excellent for showing the relief of the back beach area, and the type and density of the growth of the area.

FIG. XIC-8 - APPEARANCE OF CORAL REEFS
Note waves breaking on the edge of the reef. The channel through the reef is caused by the fresh water run-off retarding the coral growth, which in part depends upon the salinity of sea water. Depths over these reefs are best determined by stereoscopic methods.




FIG. VI C-9 - WAVE REFRACTION; An indication of bottom topography

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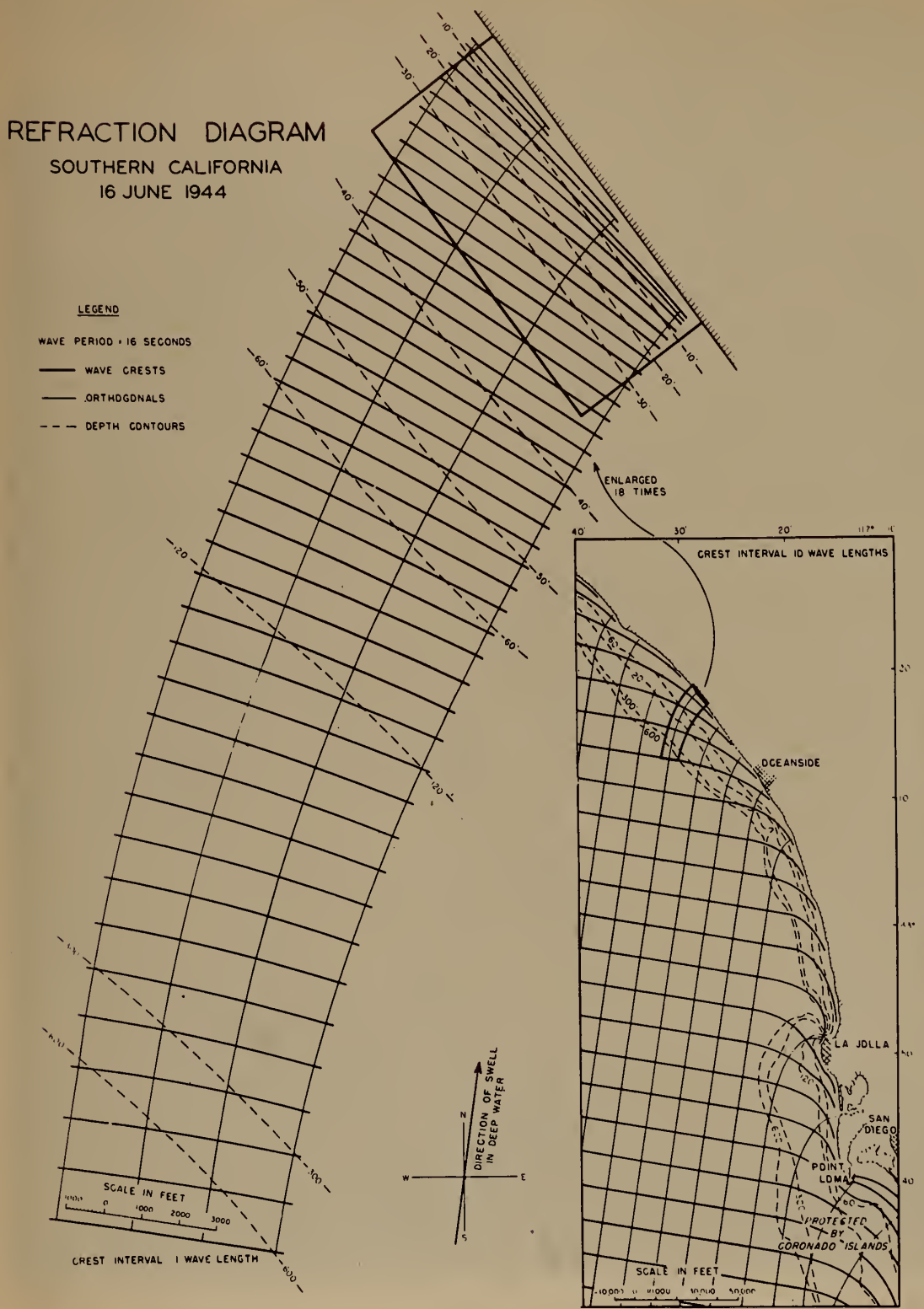
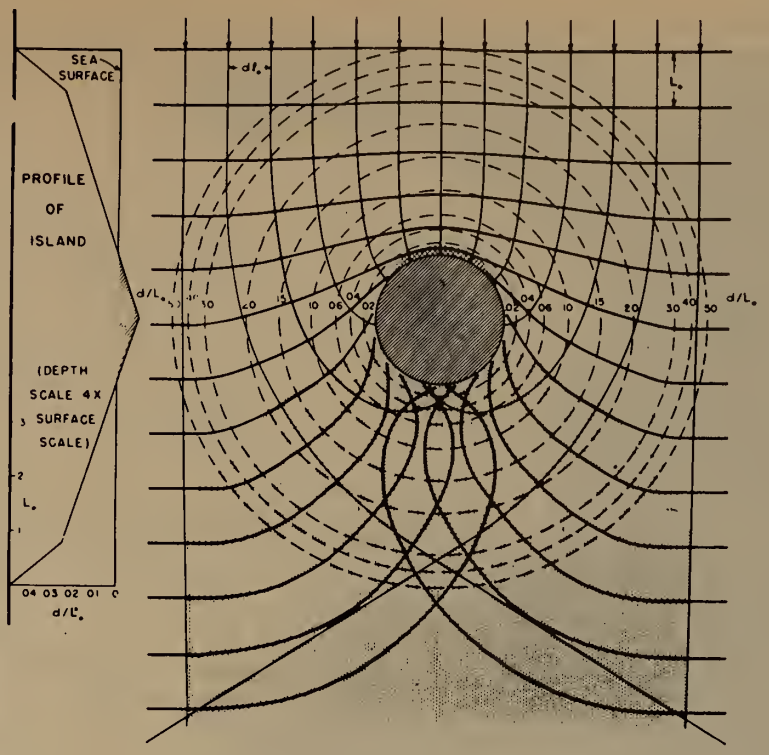
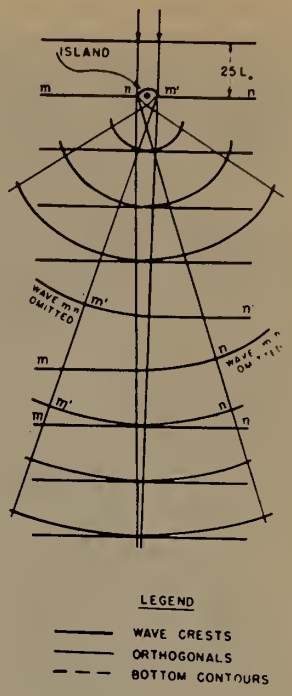


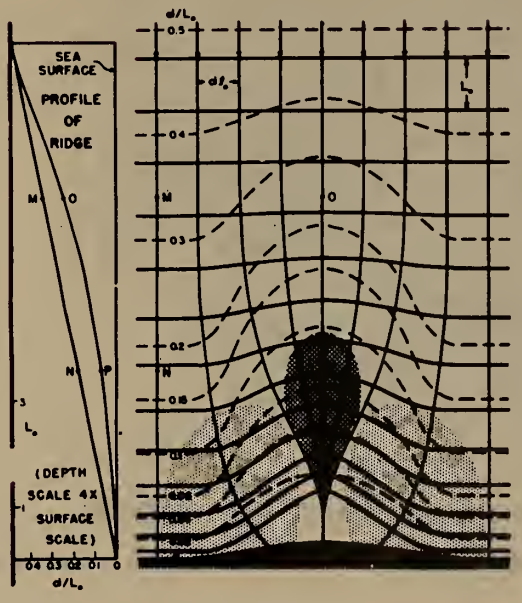

FIGURE VIIC-10



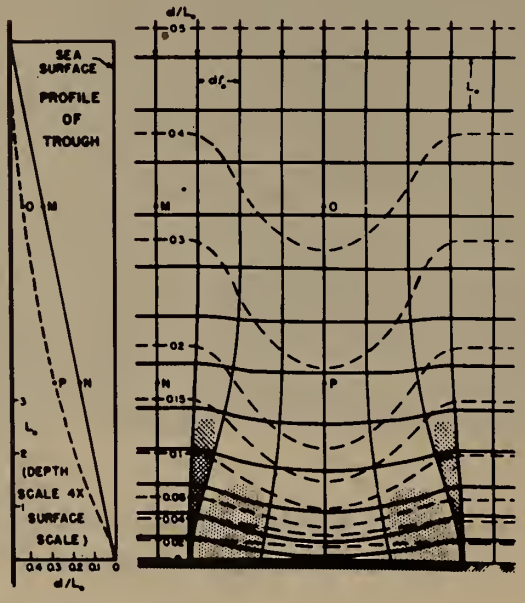

a.




b.




c.




d.

FIG. VIC-11 - REFRACTION DIAGRAMS OF AN IDEALIZED ISLAND, RIDGE AND CANYON



FIG. VIC-12 - AERIAL PHOTOGRAPH SHOWING WAVE REFRACTION, SAN CLEMENTE ISLAND

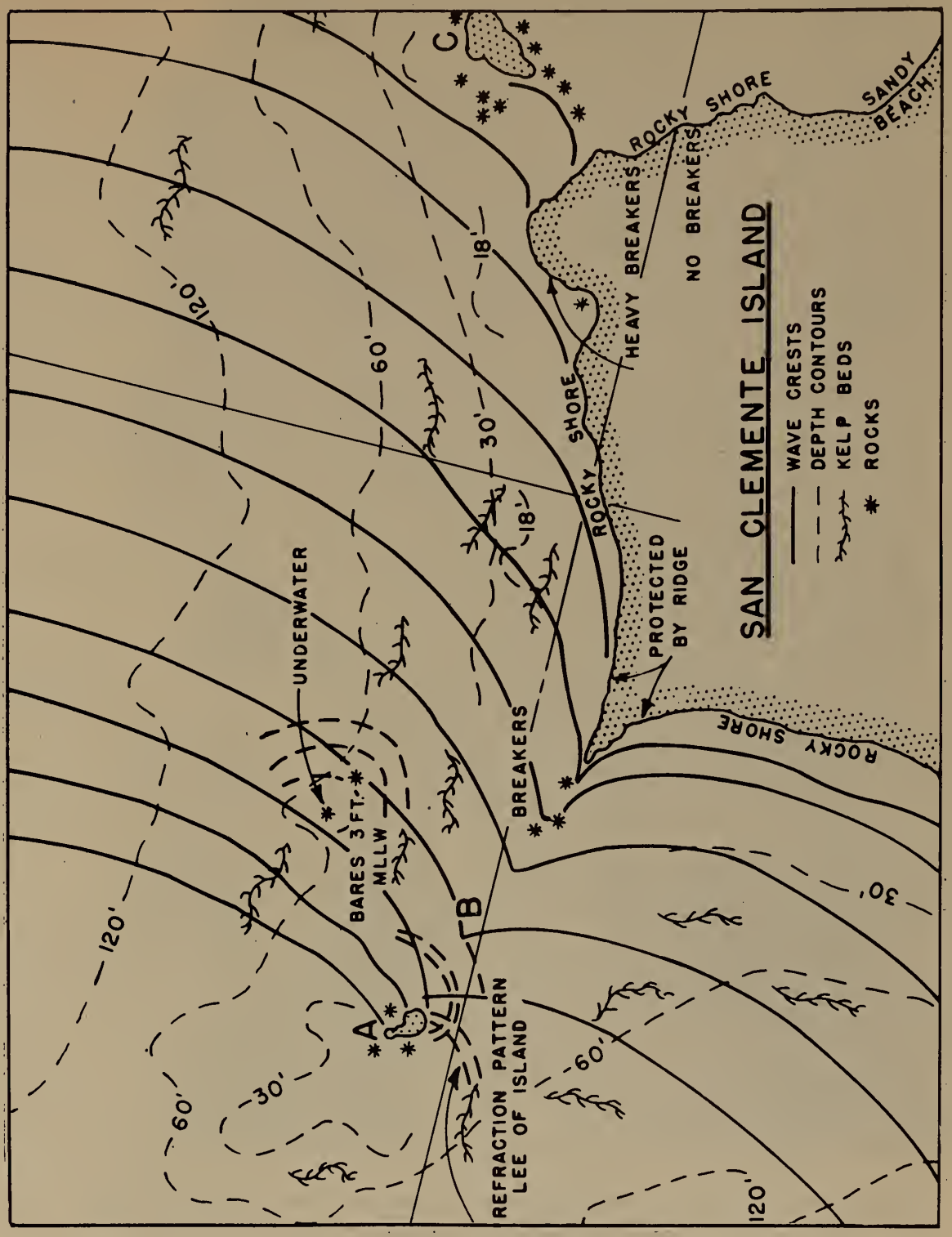



FIG. VIC-13 - REFRACTION DIAGRAM FOR CONDITIONS IN PREVIOUS FIGURE



FIG. VIC-14 - AERIAL PHOTOGRAPH SHOWING WAVE REFRACTION, SAN CLEMENTE ISLAND

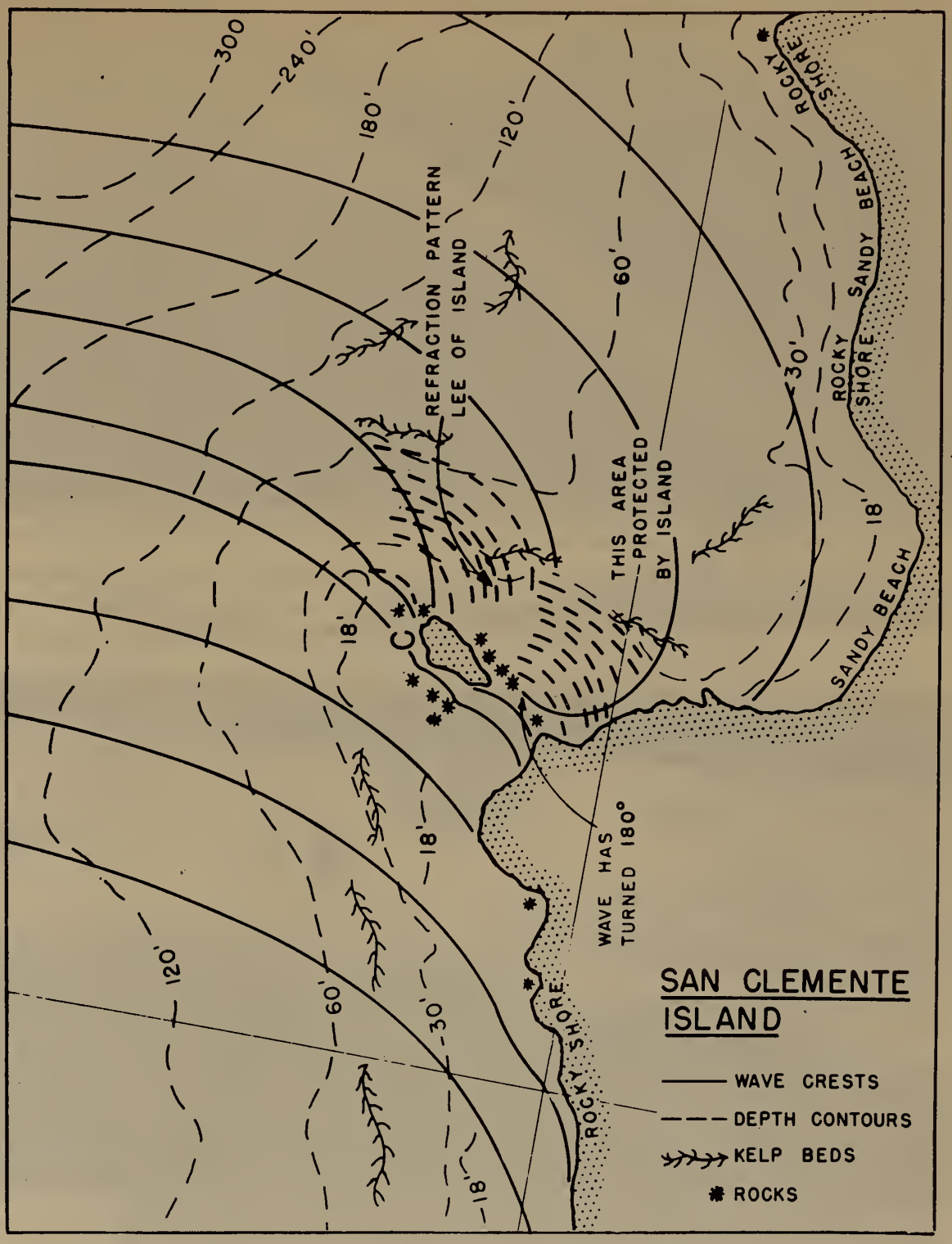



FIG. VIC-15 - REFRACTION DIAGRAM FOR CONDITIONS IN PREVIOUS FIGURE



FIGURE VIC-16 - Excellent example of a photograph which shows the presence of a bar by the breaking of the wave over the bar. In the above picture it can be seen that the breaking is not continuous; this may or may not indicate channels through the bar. Many more photos of the area would have to be analyzed to determine this.

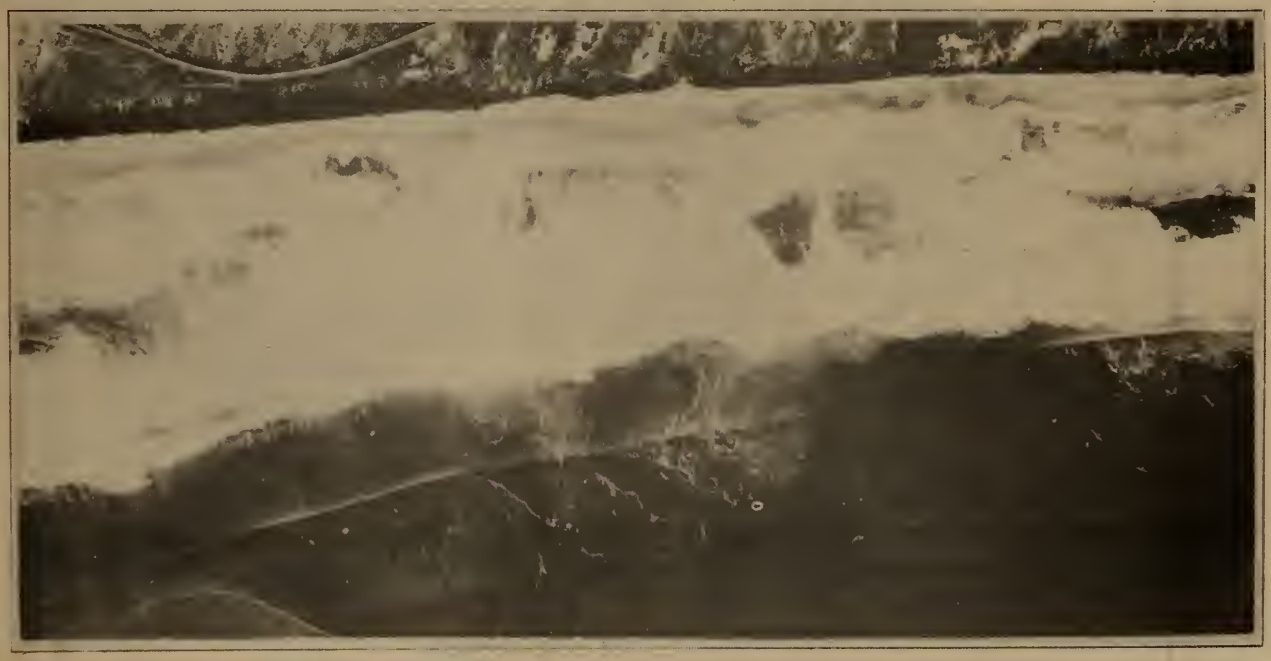


FIGURE VIC-17 - Vertical photograph of wave breaking over bar. Note rip currents and peaking-up of wave just prior to breaking.

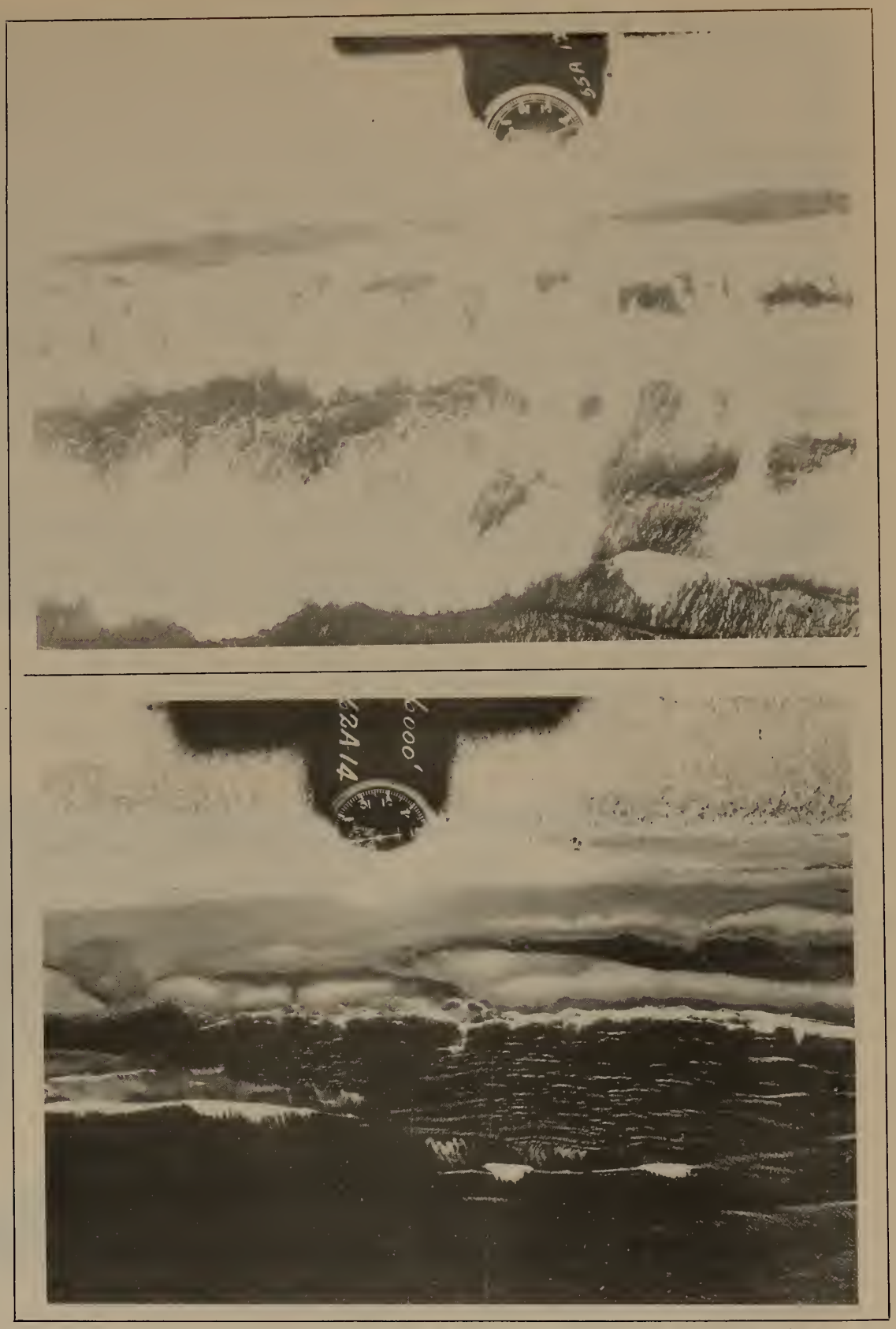


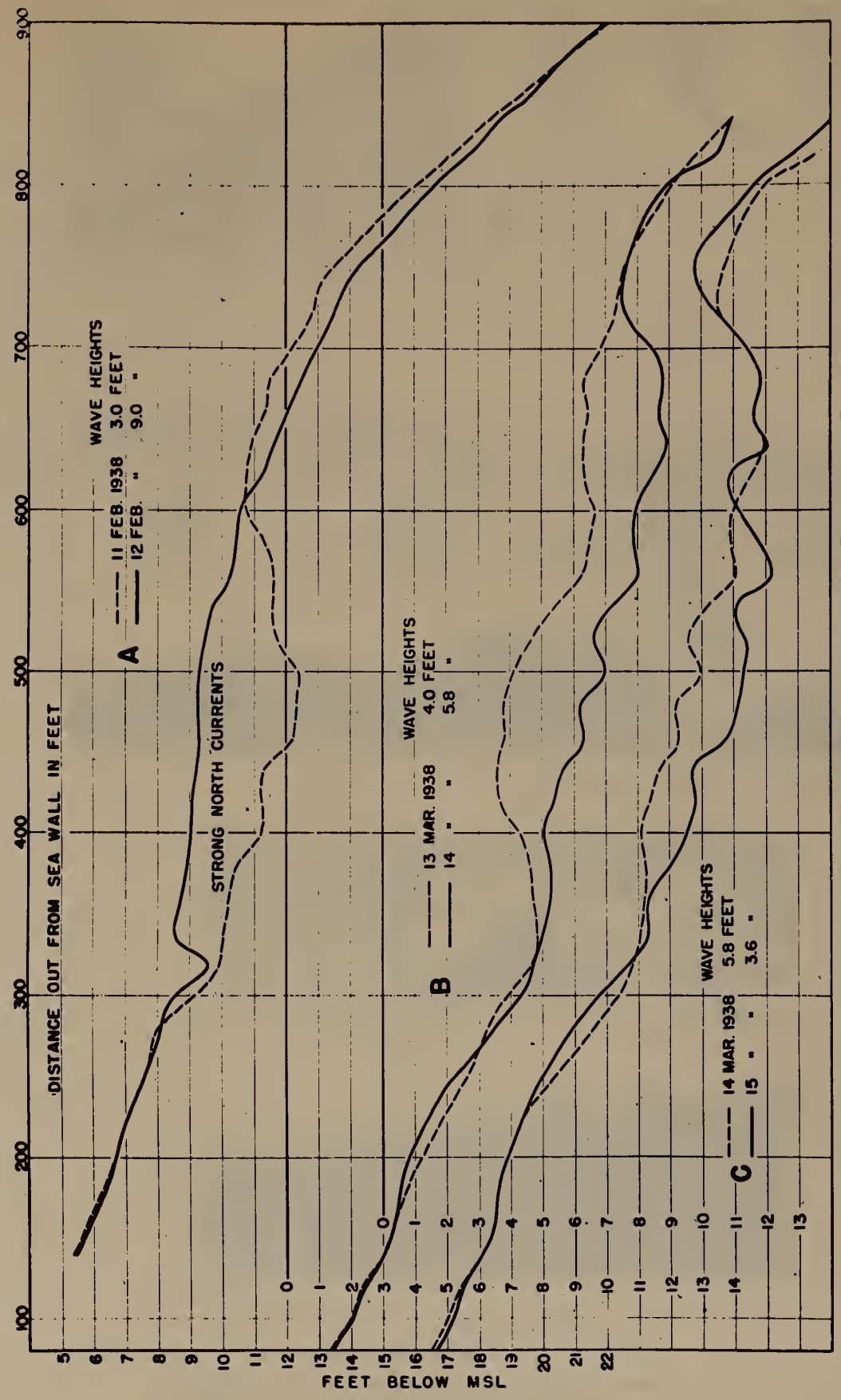
FIGURE VIC-18 - Low tide exposes bars that are indicated at high tide.



FIGURE VI C-19 - Clear water photograph which shows the shoal water as light areas. Also very good picture showing the rock outcrops. The bay and point give very good reference point with which to measure positions.



FIGURE VI C-20 - Example of light area caused by wave action putting material into suspension. It will be noticed the line between dark and light areas is very clearly defined, as compared to Fig. VI C-19.




A-PROFILE CHANGES AT SCRIPPS PIER DURING STRONG NORTH CURRENTS AND LARGE WAVES
B-EXCAVATION AT SCRIPPS PIER DURING INCREASED WAVE HEIGHT
C-BUILDING UP OF BAR OUTSIDE AT SCRIPPS PIER DURING DECREASED WAVE HEIGHT

FIG. VIC-21 - BAR SHIFT DURING 24 HOURS

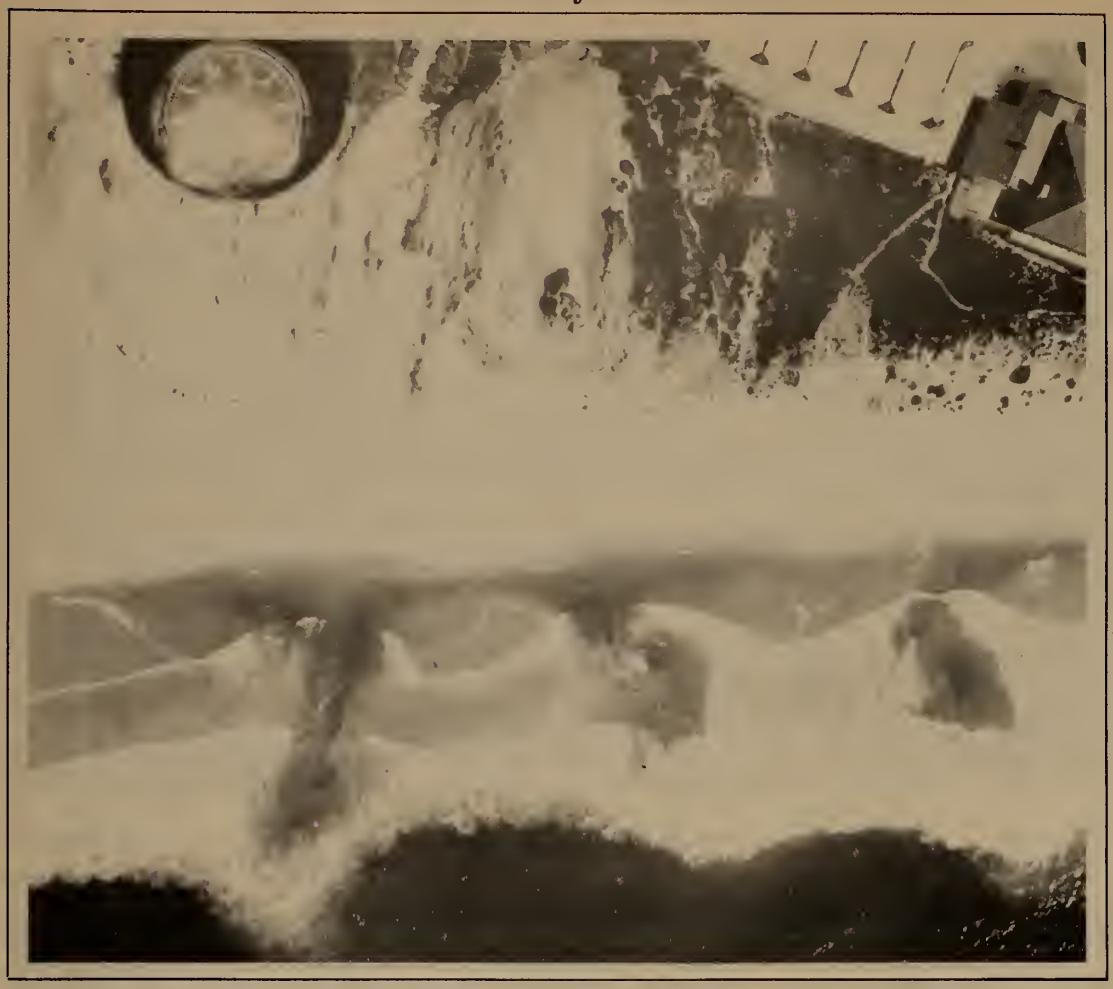


FIGURE VIC-22 - Rip currents



FIGURE VIC-23 - Same area as in Fig. VIC-22. Note lack of rip currents; tide stage about the same. Note pipe outfall at left of both pictures.

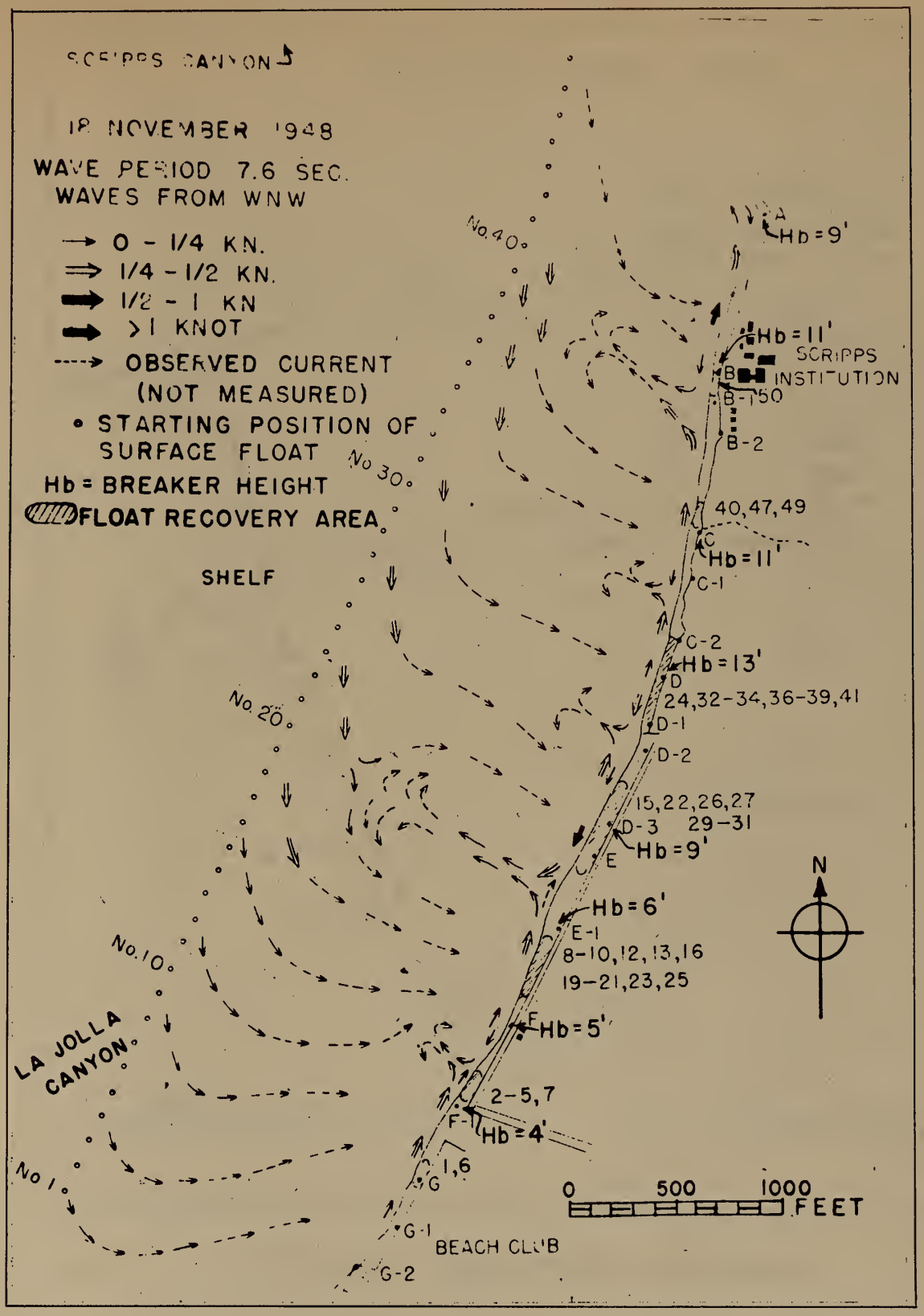



FIG. VIC-24 - NEAR-SHORE CIRCULATION

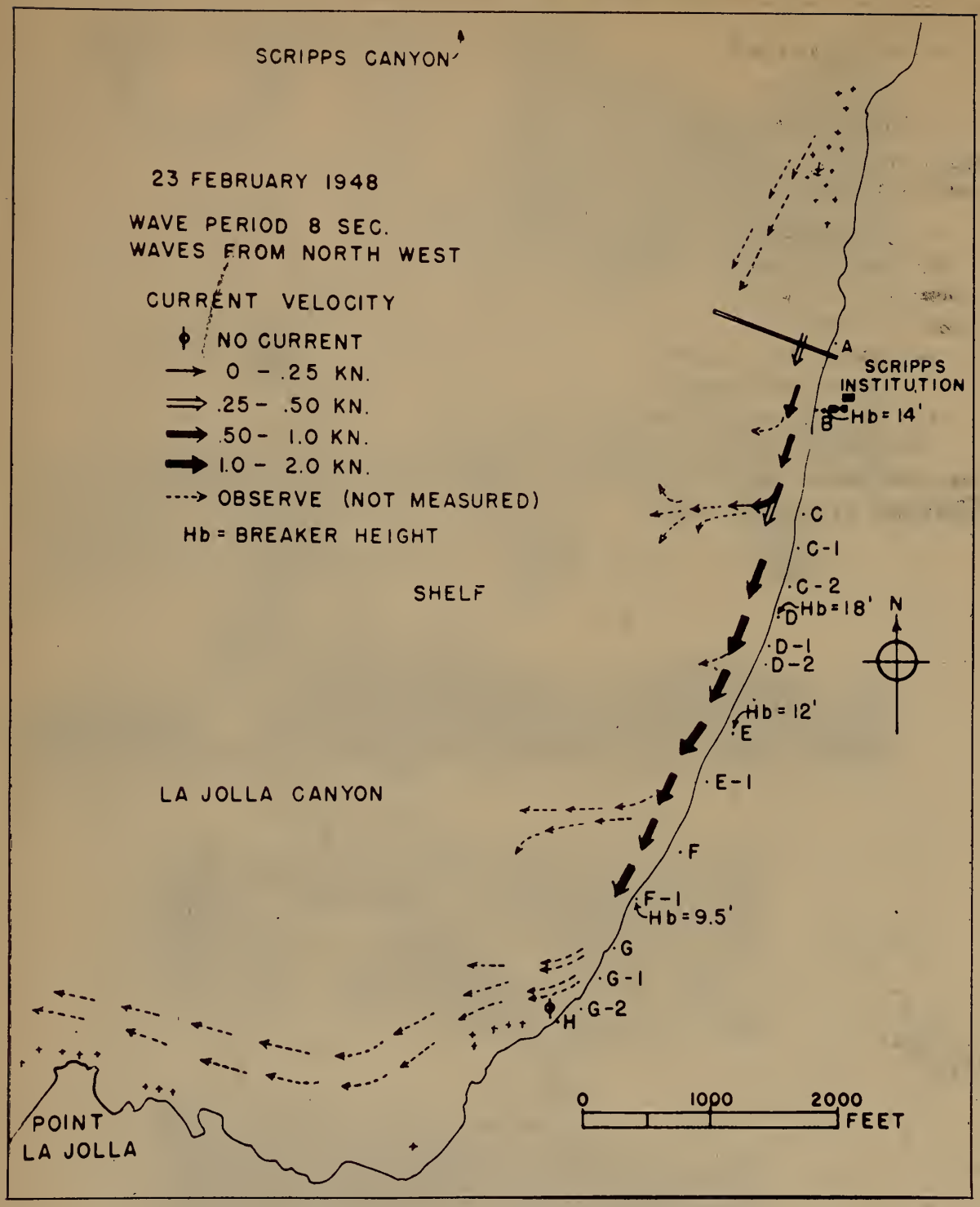



FIG. VIC-25 - NEAR-SHORE CIRCULATION

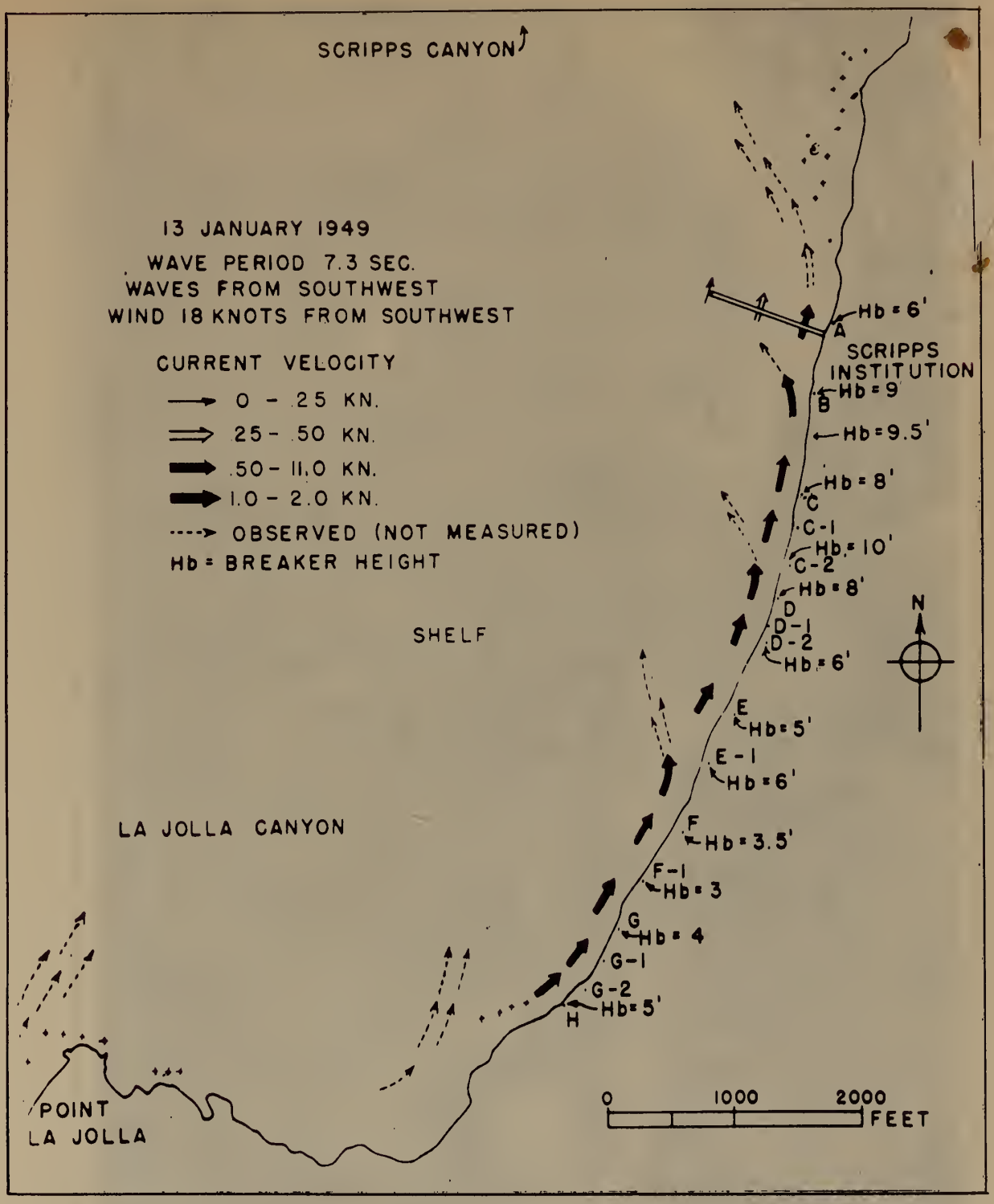


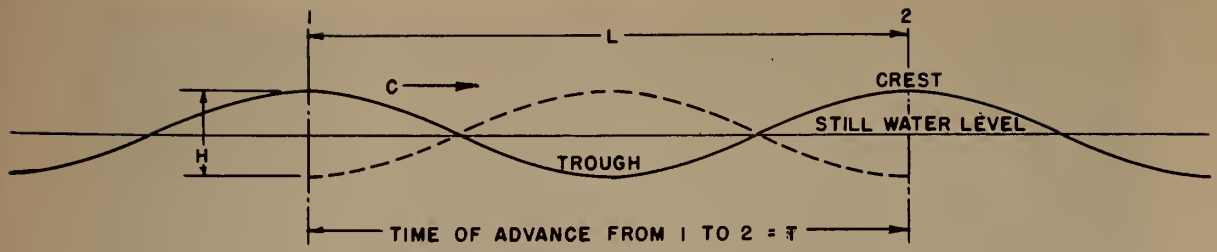

FIG. VIC-26 - NEAR-SHORE CIRCULATION

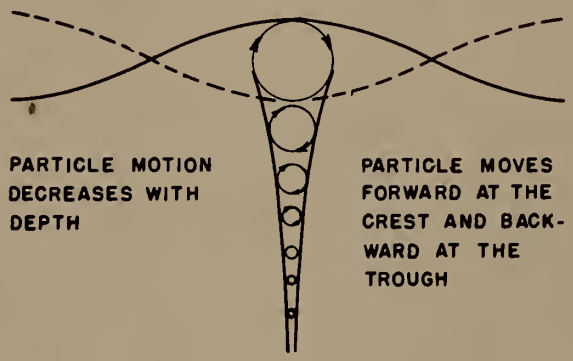


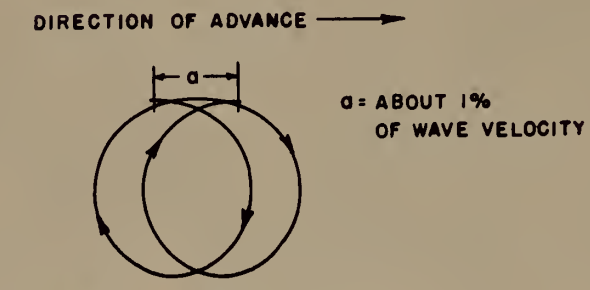

FIG. VIC-27 - LOCATION OF UNDERWATER OBSTACLES




FIG. VIC-28 - LOCATION OF UNDERWATER OBSTACLES



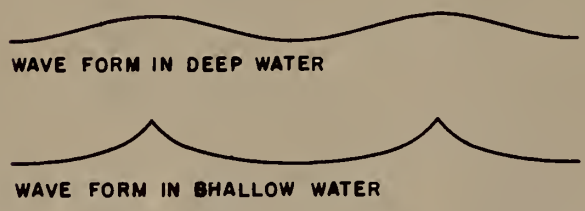




FIGURE VIC-29



FIGURE VIC-30 - Long Period Swell; note the change in crest configuration as crests move into shallow water.

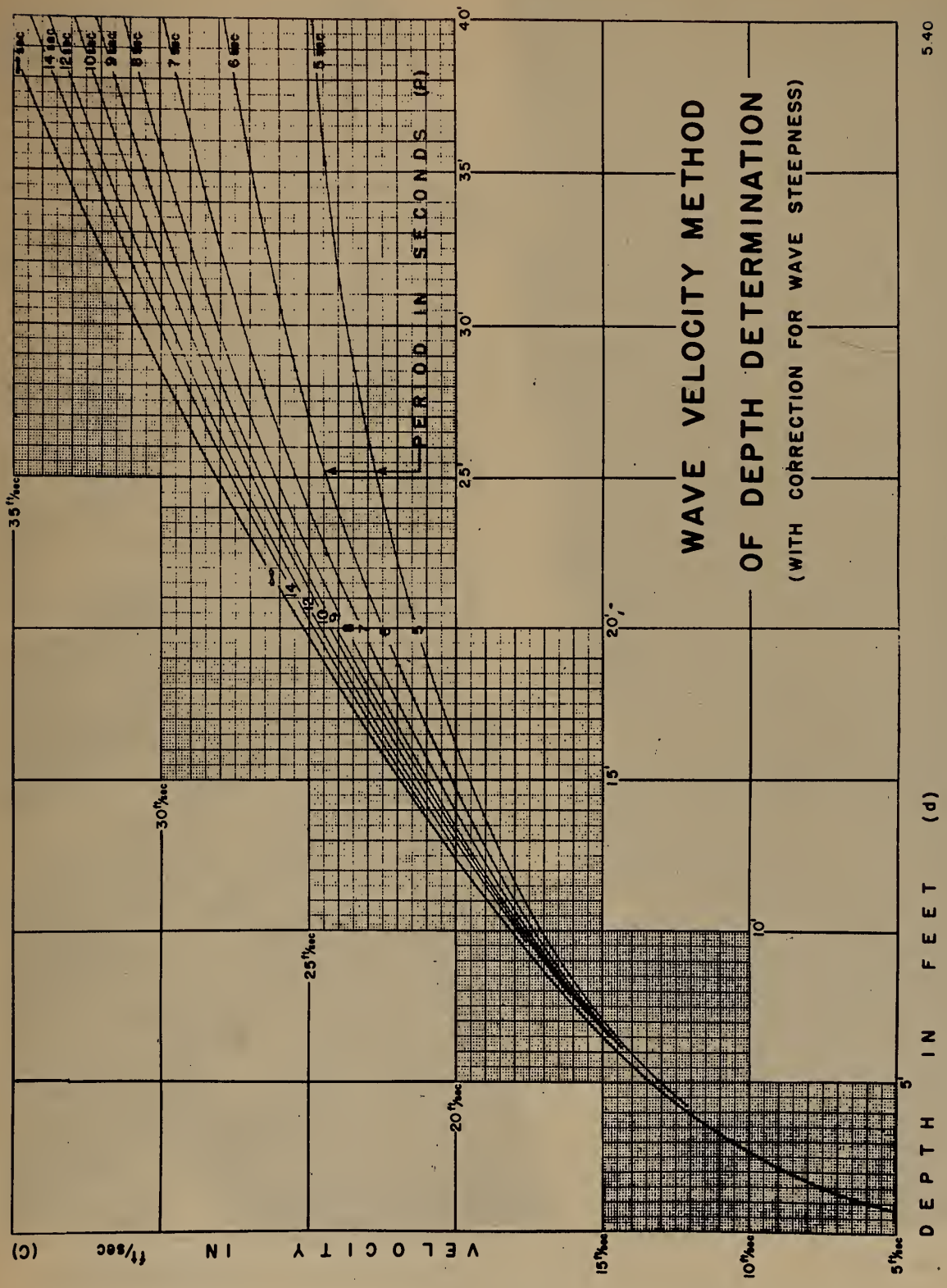



FIGURE VIC-31

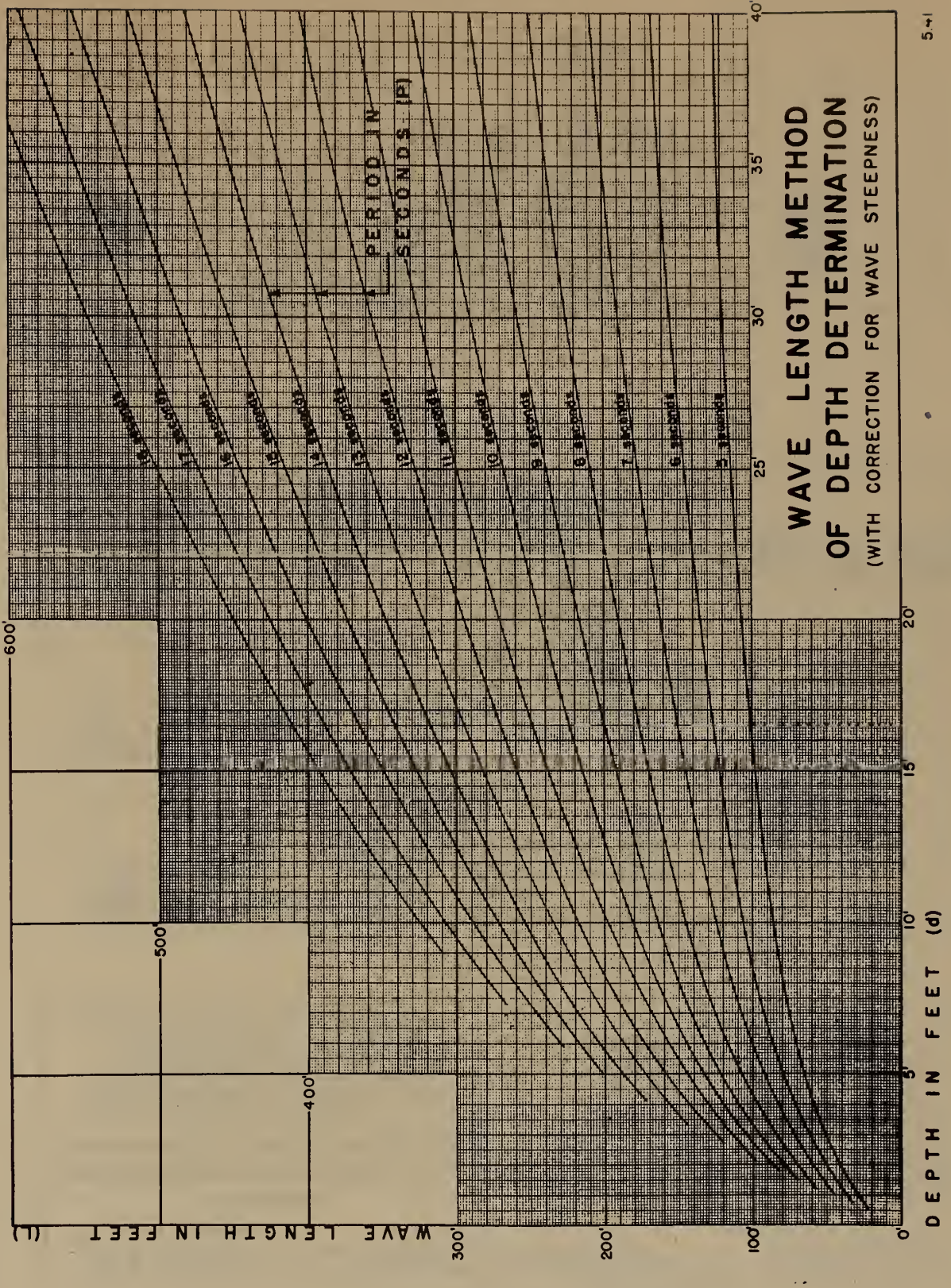
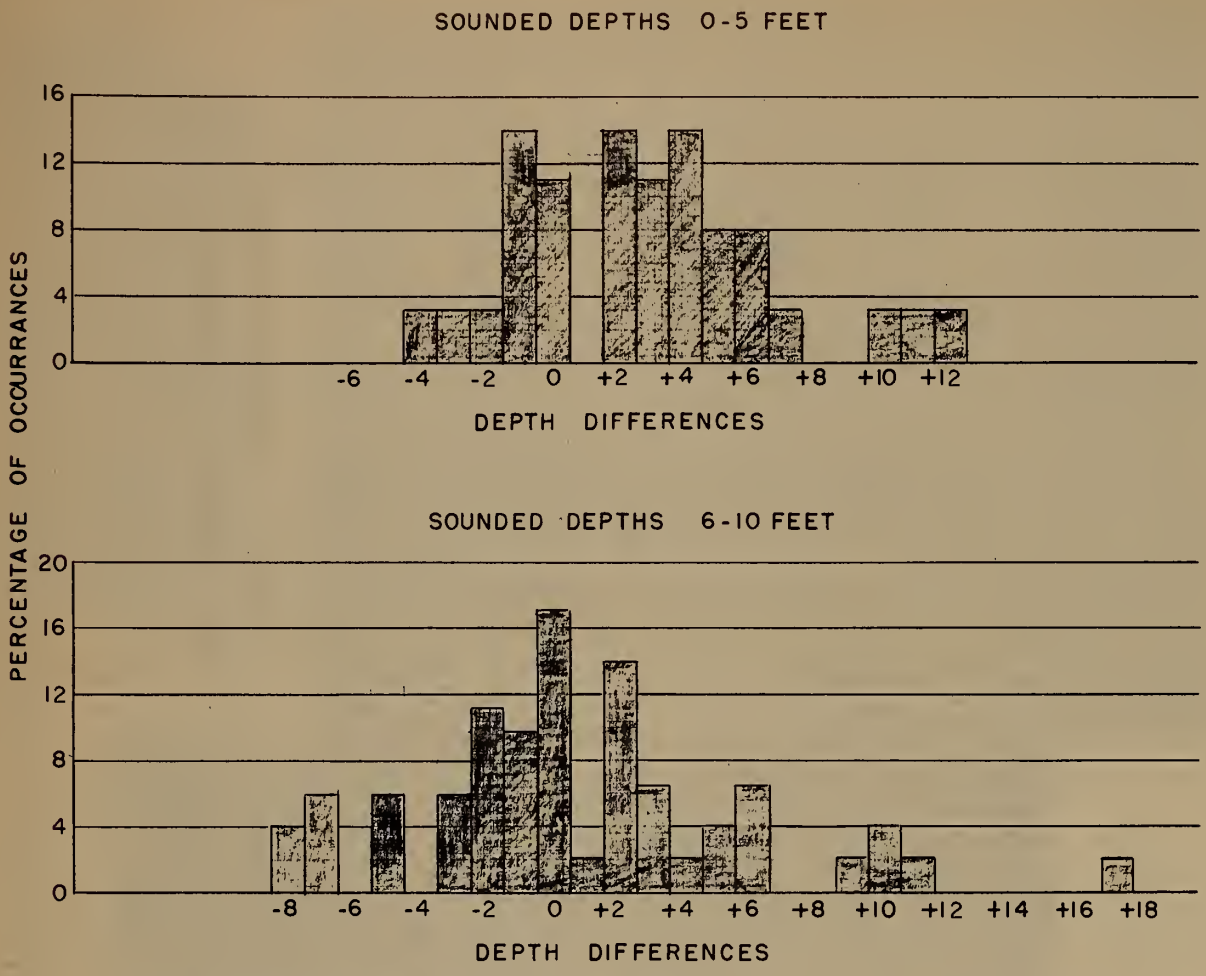



FIGURE VIC-32




– Sounded Depth greater than Calculated Depth
\+ Sounded Depth less than Calculated Depth

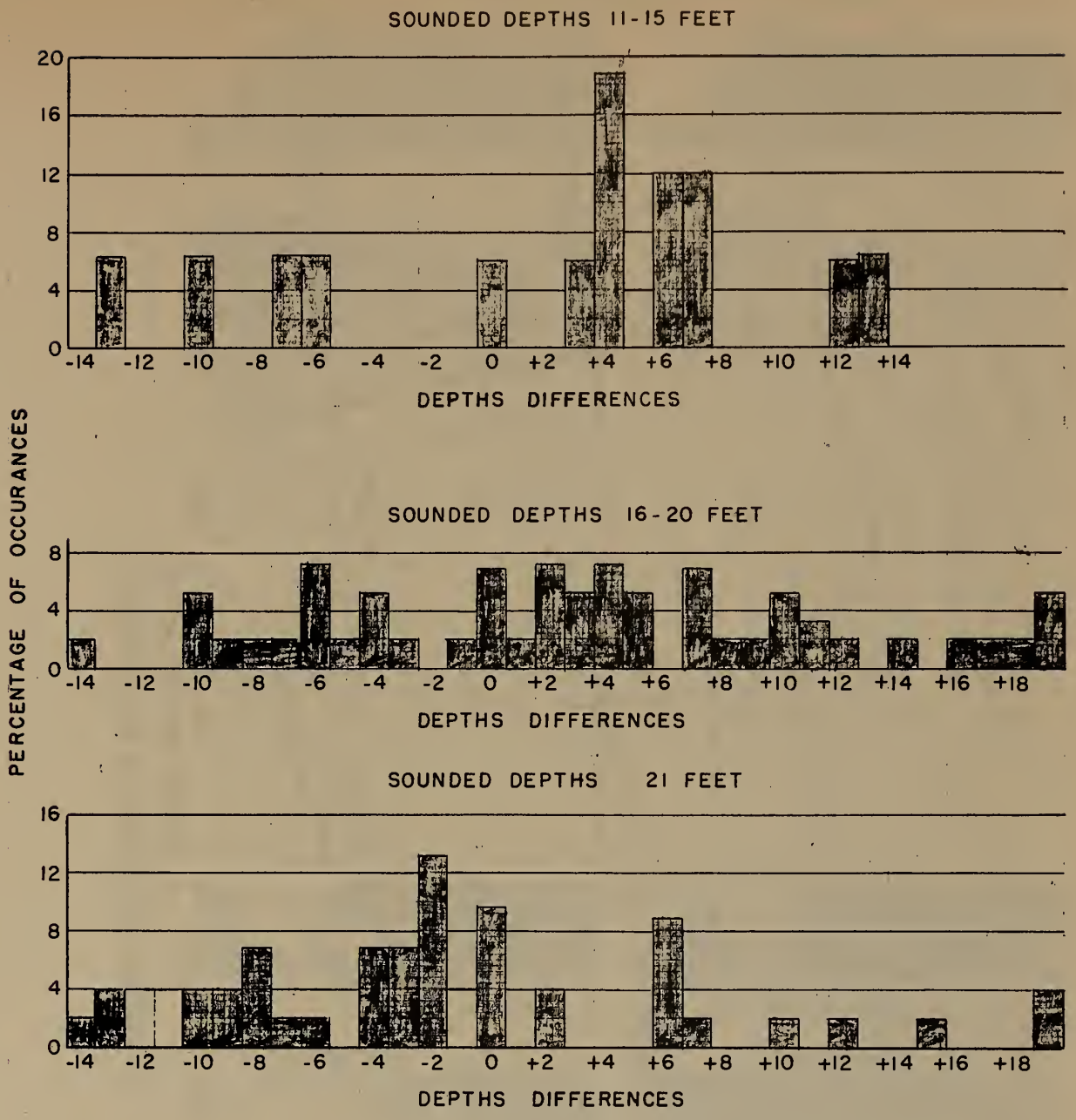
FREQUENCY DISTRIBUTIONS
OF DEPTH DIFFERENCES
OBTAINED BY SOUNDINGS
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FIGURE VIC-33




– Sounded Depth greater than Calculated Depth

\+ Sounded Depth less than Calculated Depth

FREQUENCY DISTRIBUTIONS OF DEPTH DIFFERENCES OBTAINED BY SOUNDINGS AND AERIAL PHOTOGRAPHS

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FIGURE VIC-34

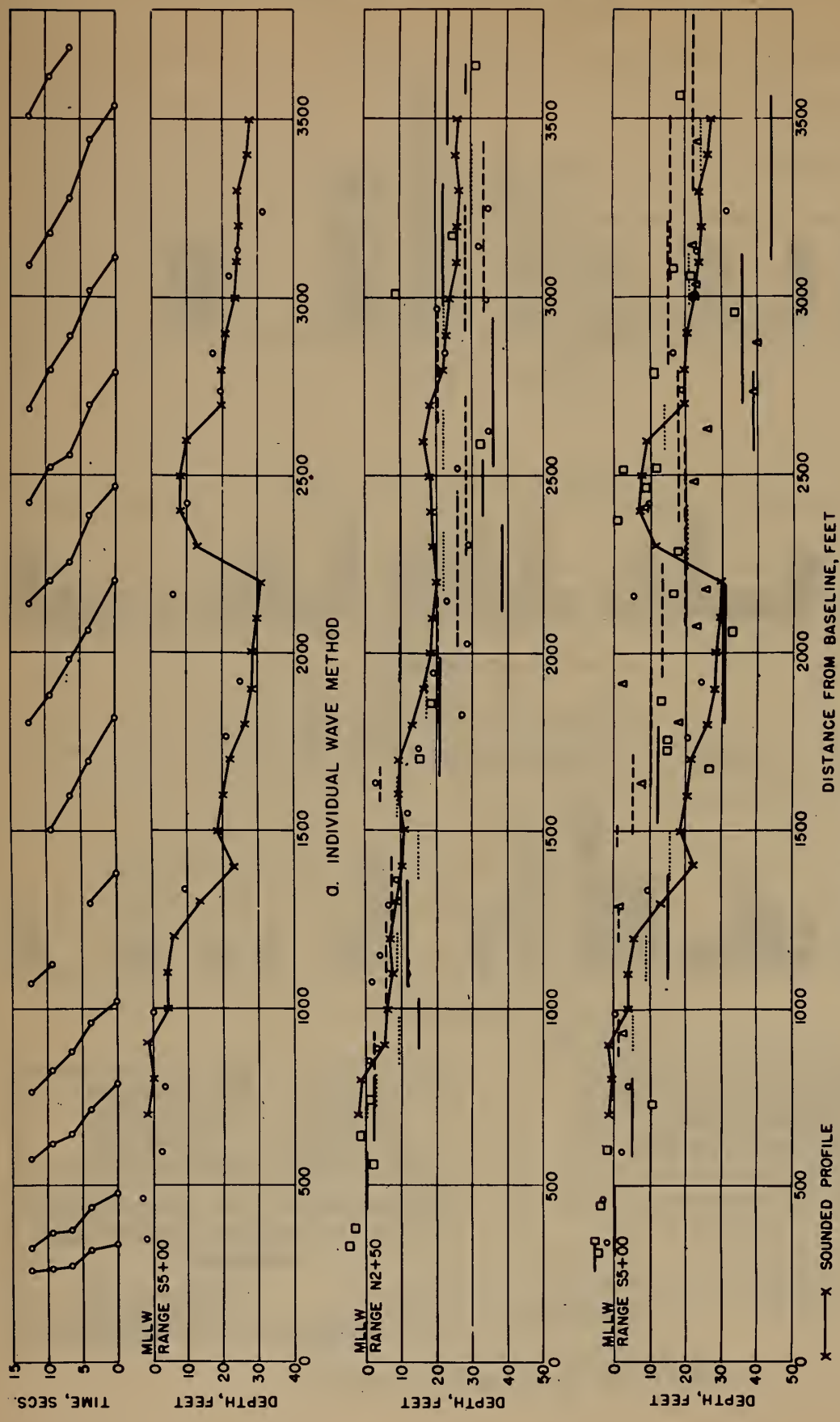



FIGURE VII C-35

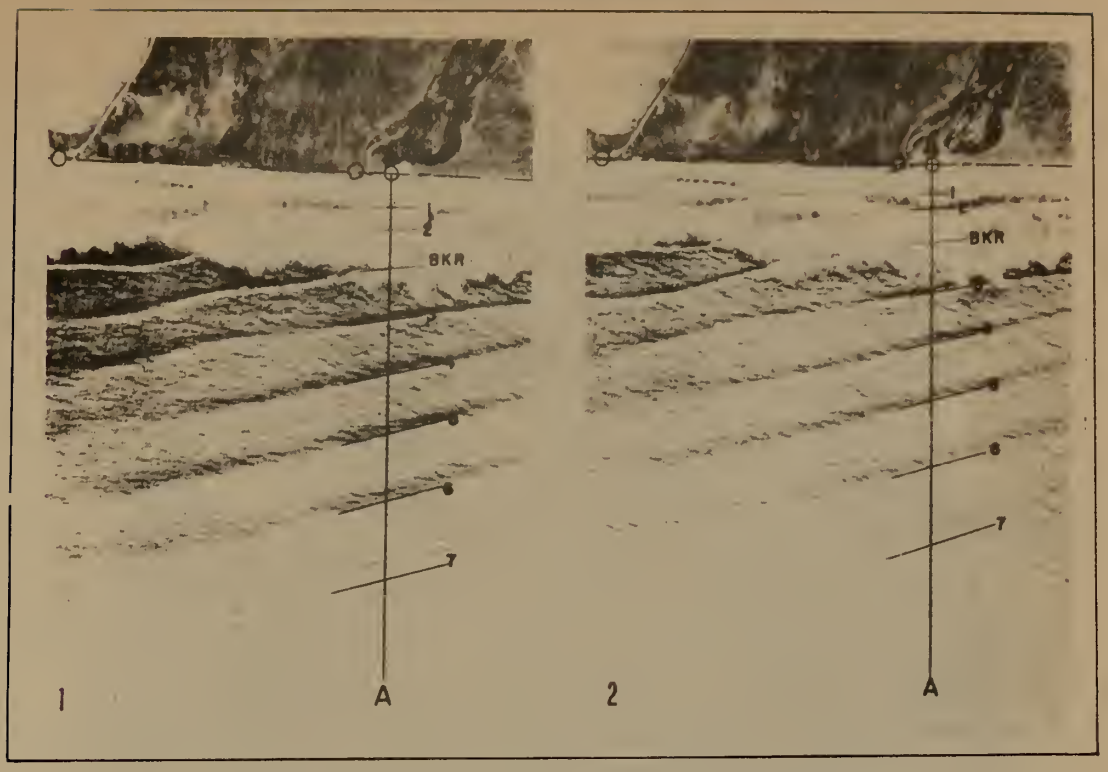



FIG. VIC-36 - WAVE LENGTH EXAMPLE

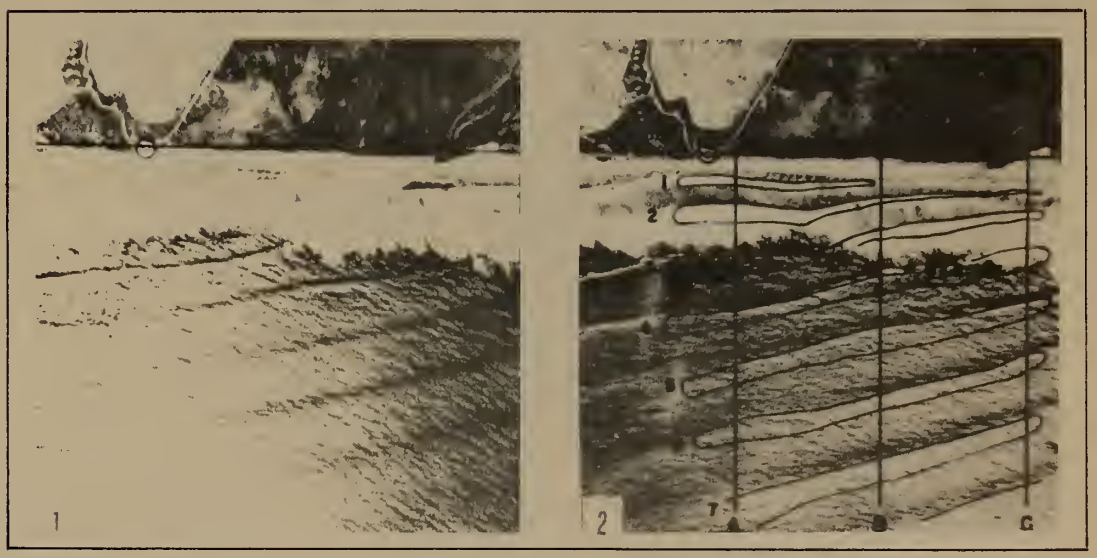



FIG. VIC-37 - WAVE VELOCITY EXAMPLE

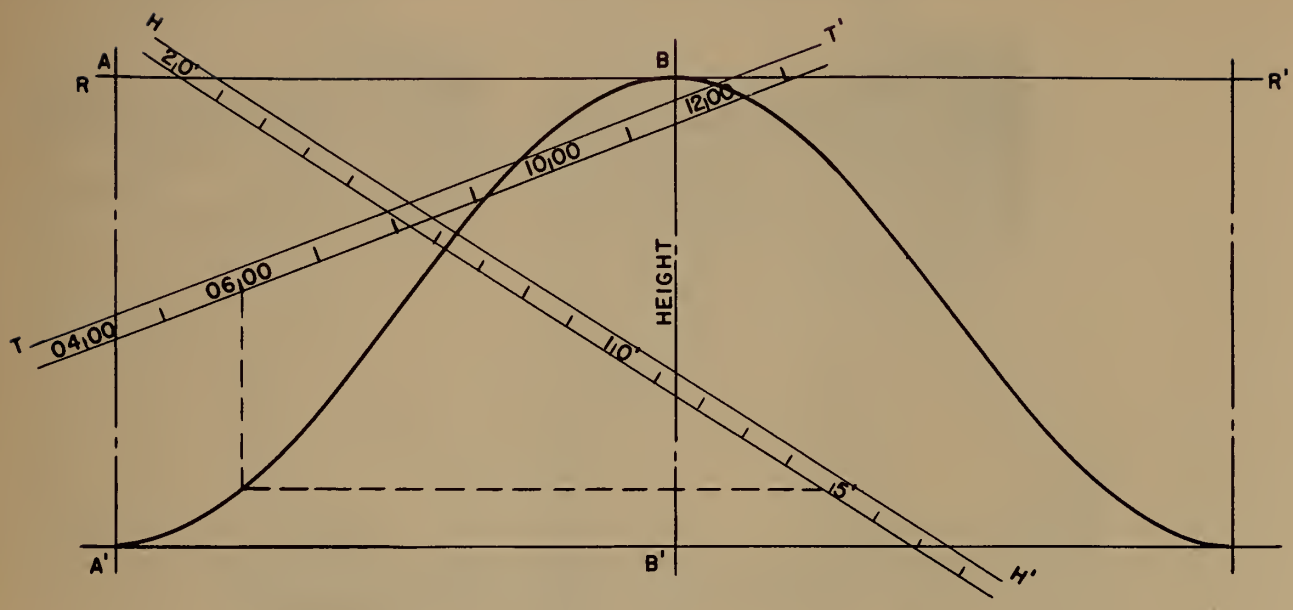



FIGURE VI C-38 - Method for timing the intermediate water lines.




FIGURE VIC-39 - SONNE CAMERA




FIGURE VIC-40 - Sample of Sonne stereo strip photograph

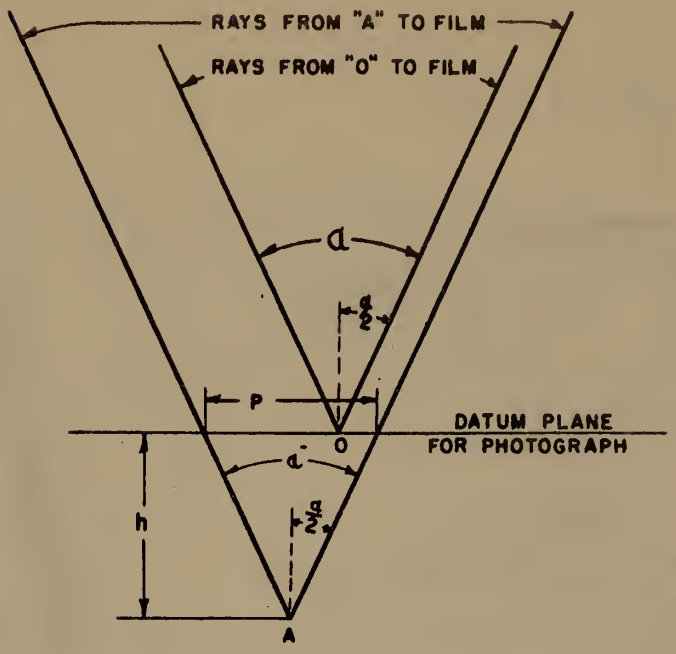



FIGURE VIC-41

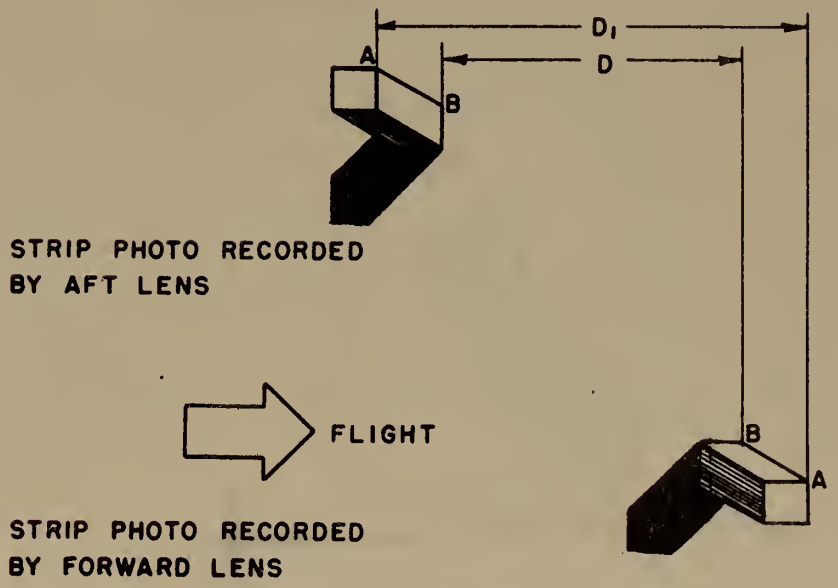
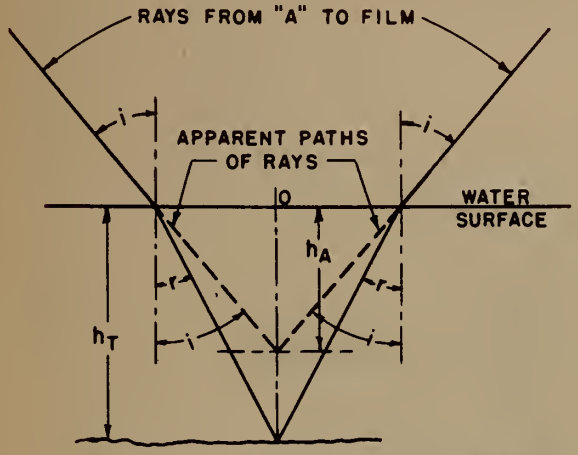



FIGURE VIC-42




$h_A$ - APPARENT DEPTH DISCOUNTING REFRACTION

$h_T$ - TRUE DEPTH

FIGURE VIC-43

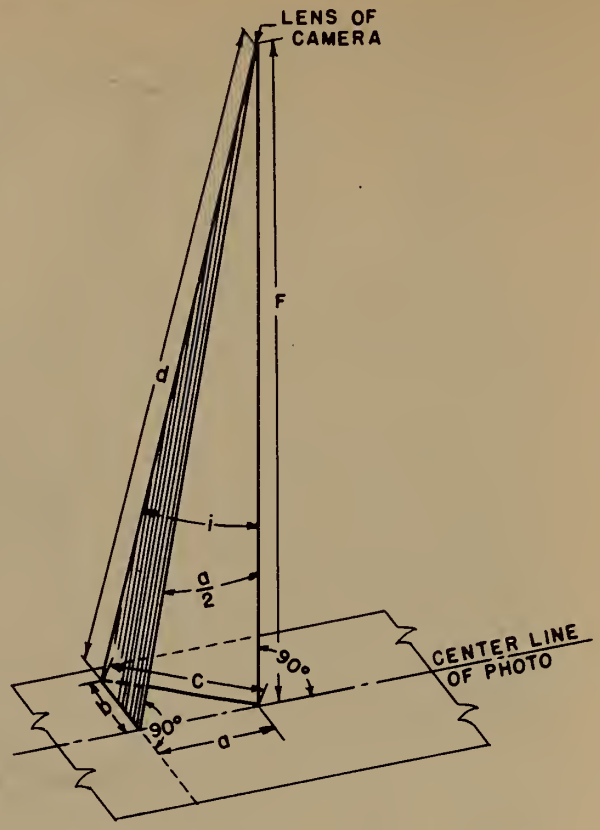



FIGURE VIC-44

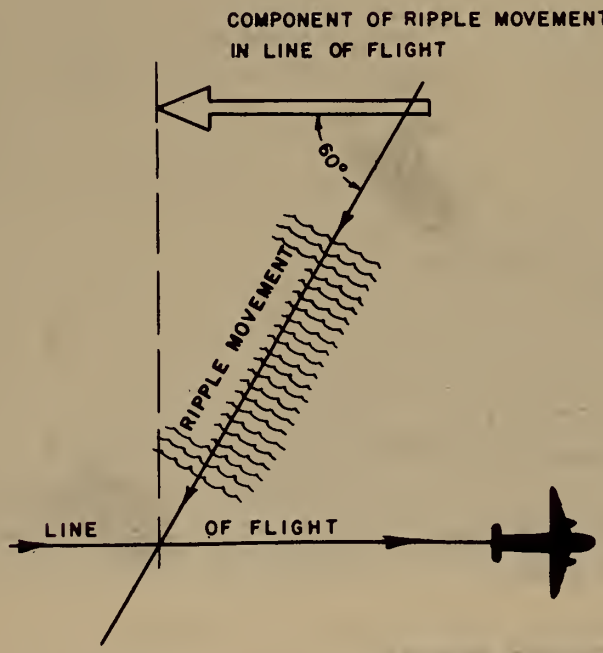



FIGURE VIC-45

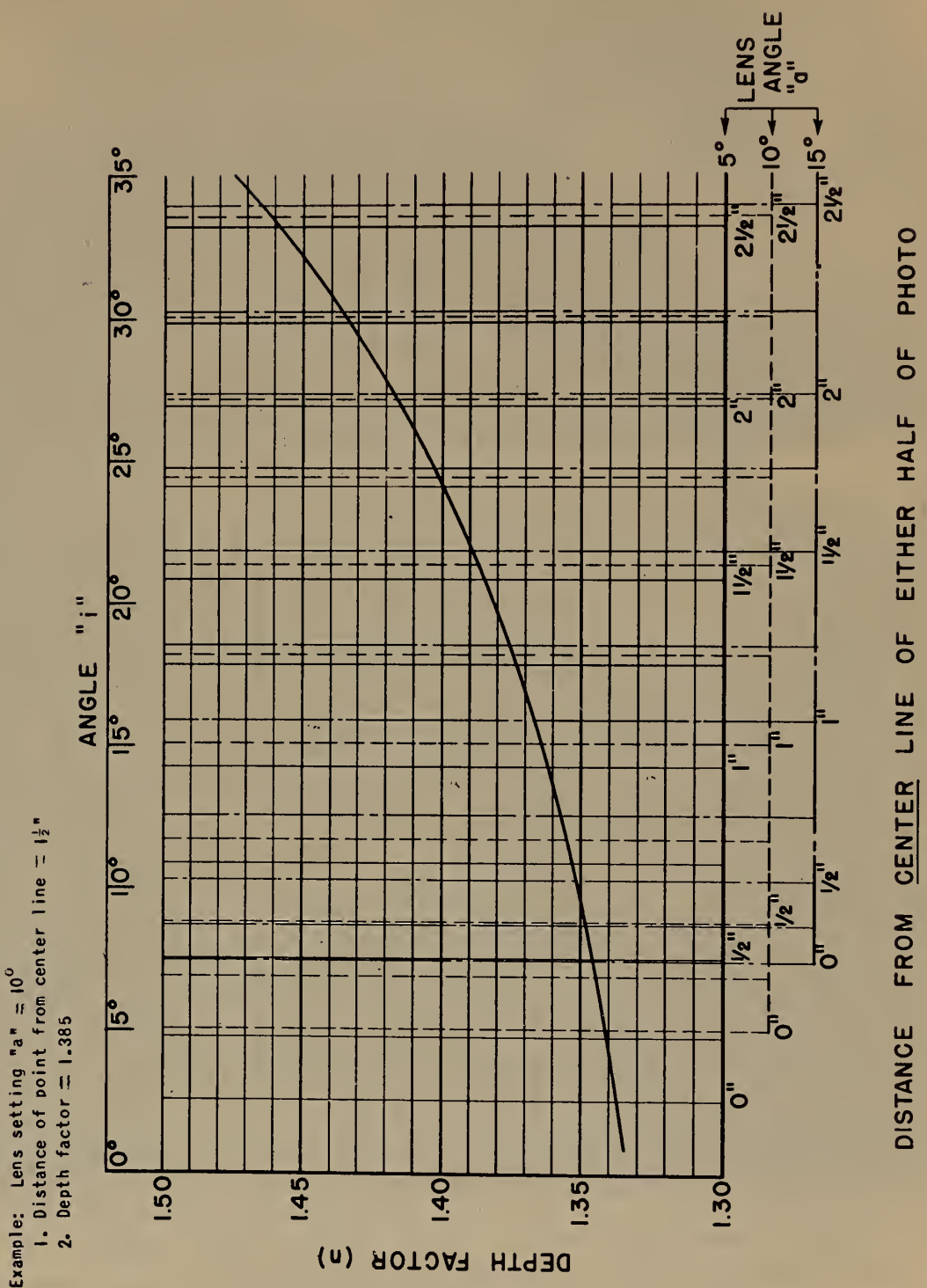



FIGURE VIC-46

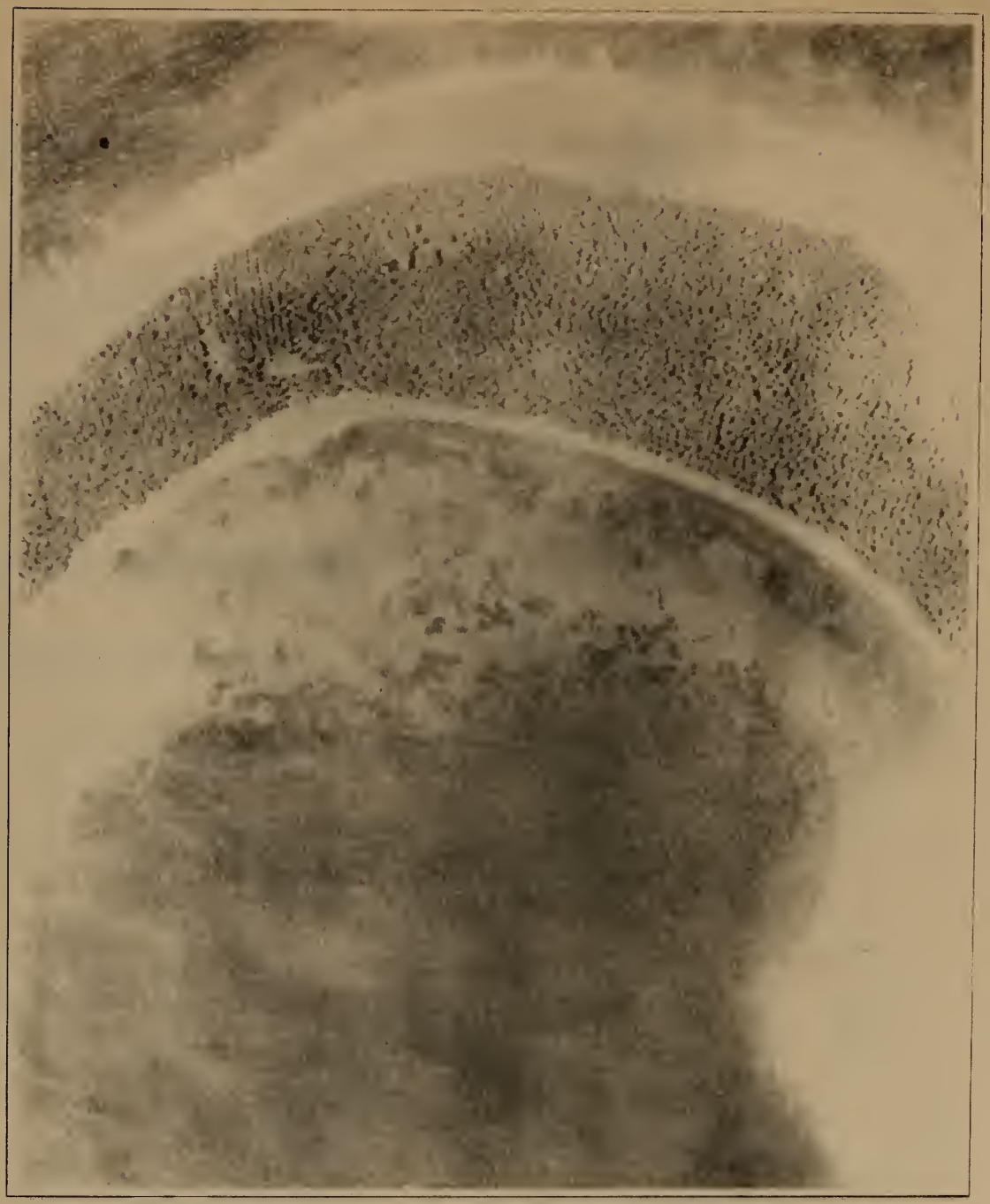


FIGURE VIC-47 - A coral area of this type would be ideal for stereoscopic depth determination. There is ample bottom detail for the Sonne stereo method and ample surface control for the standard camera method. The latter method could be used only if the beach were undefended. RF = 1:5000.

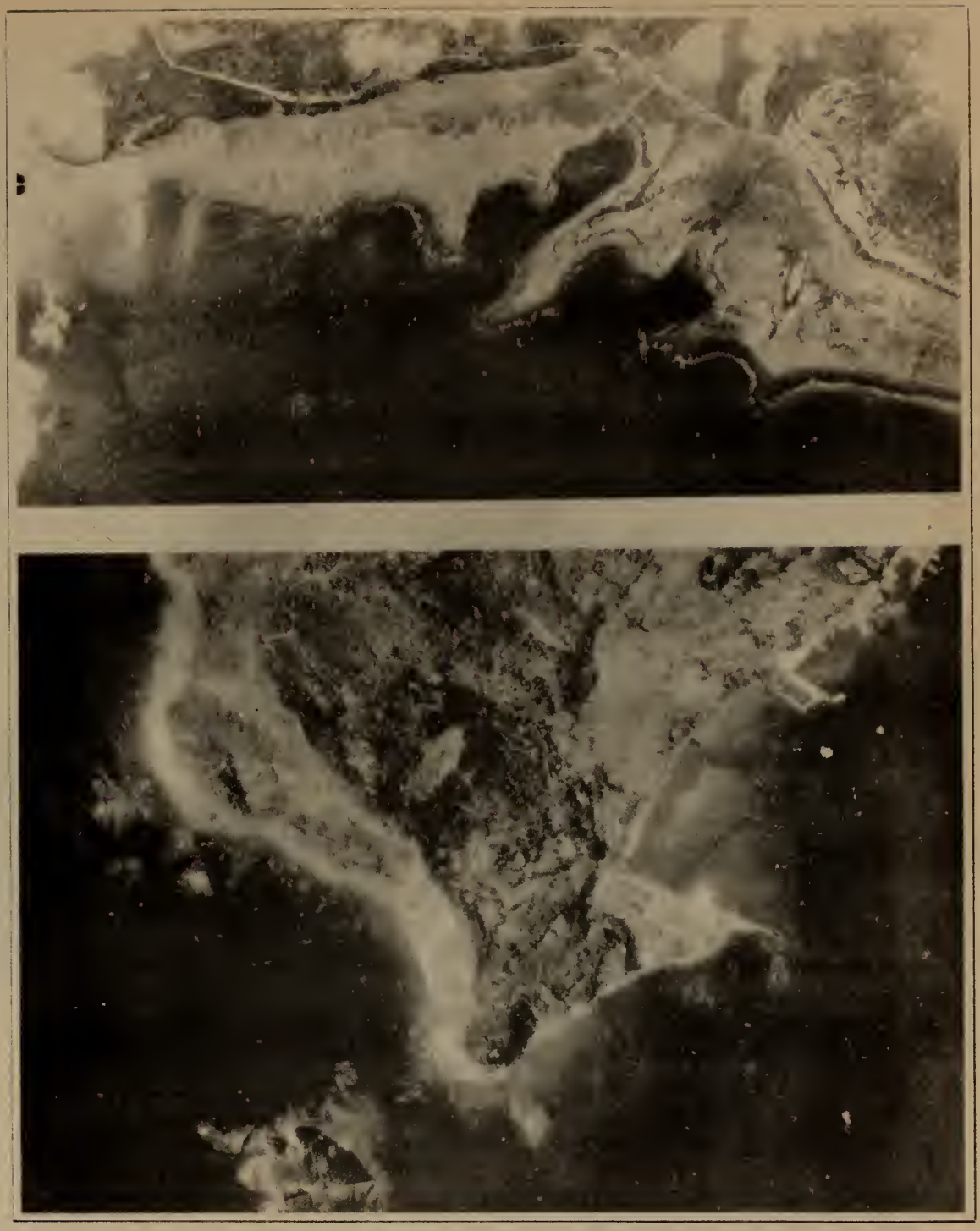


FIGURE VIC-48 - Areas of clear water and ample bottom detail illustrating ideal conditions for stereoscopic methods of depth determination, particularly the Sonne stereo strips.
RF = 1:10000

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(SIGNATURE) M H Brown (RANK) Civ

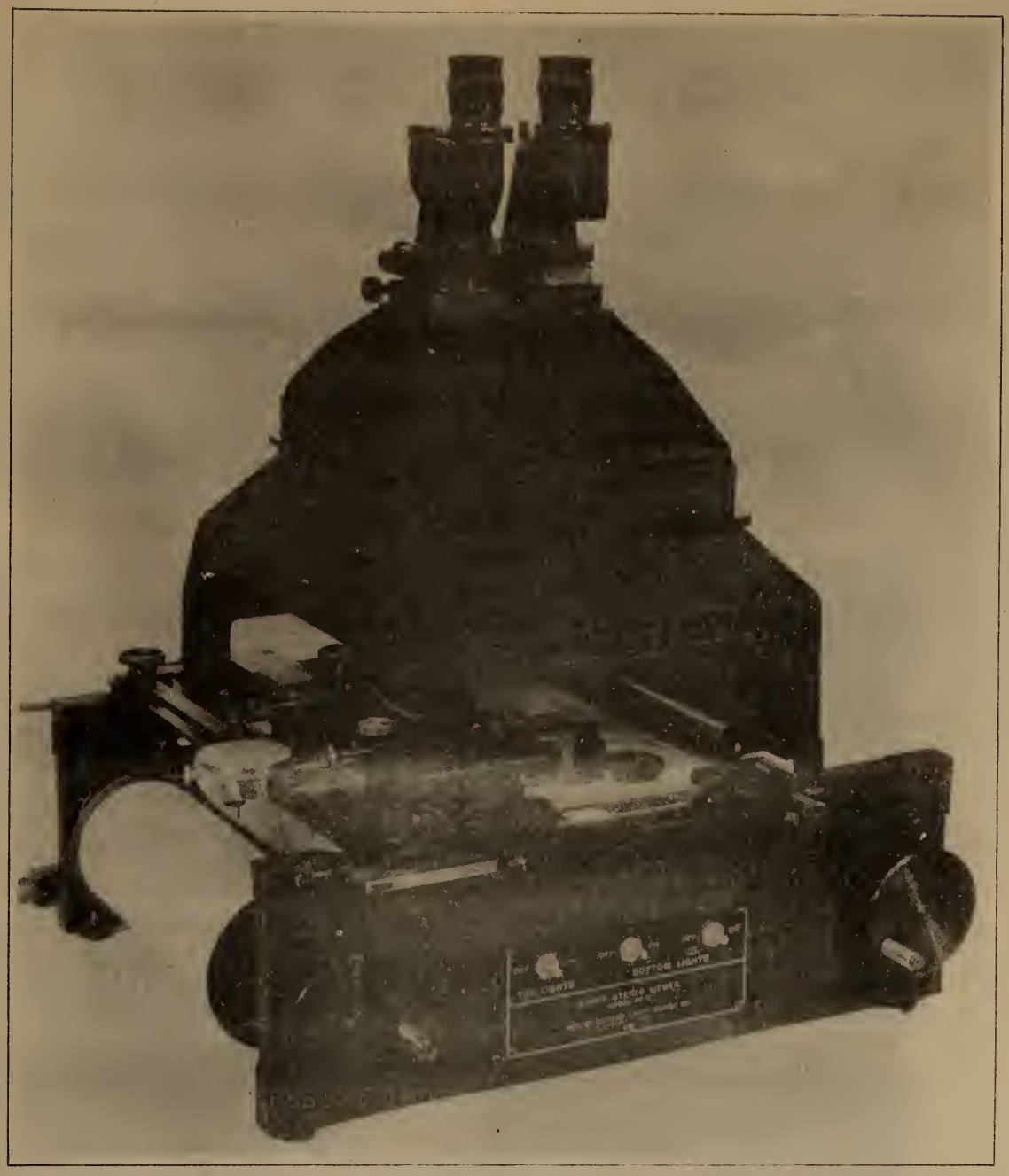


FIGURE VIC-49 - Sonne Viewer with parallax device

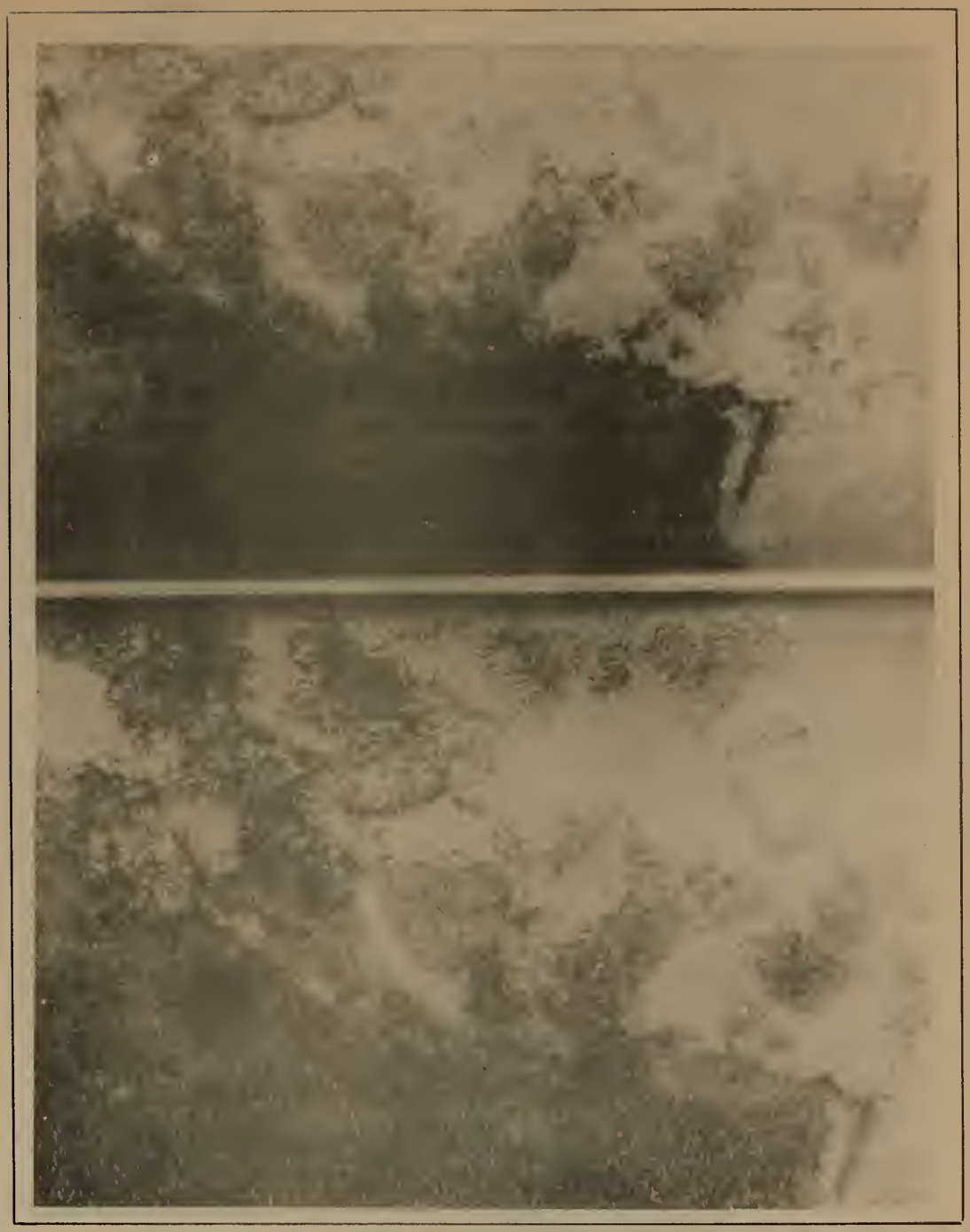


FIGURE VIC-50 - Section of a stereo photograph illustrating good bottom detail photographed under ideal conditions for depth determination. RF = 1:600

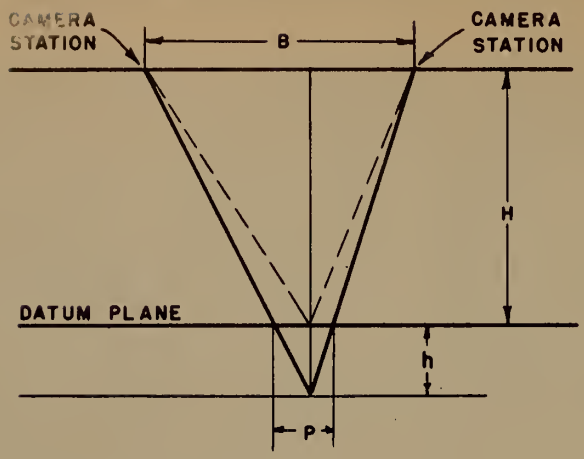



FIGURE VIC-51

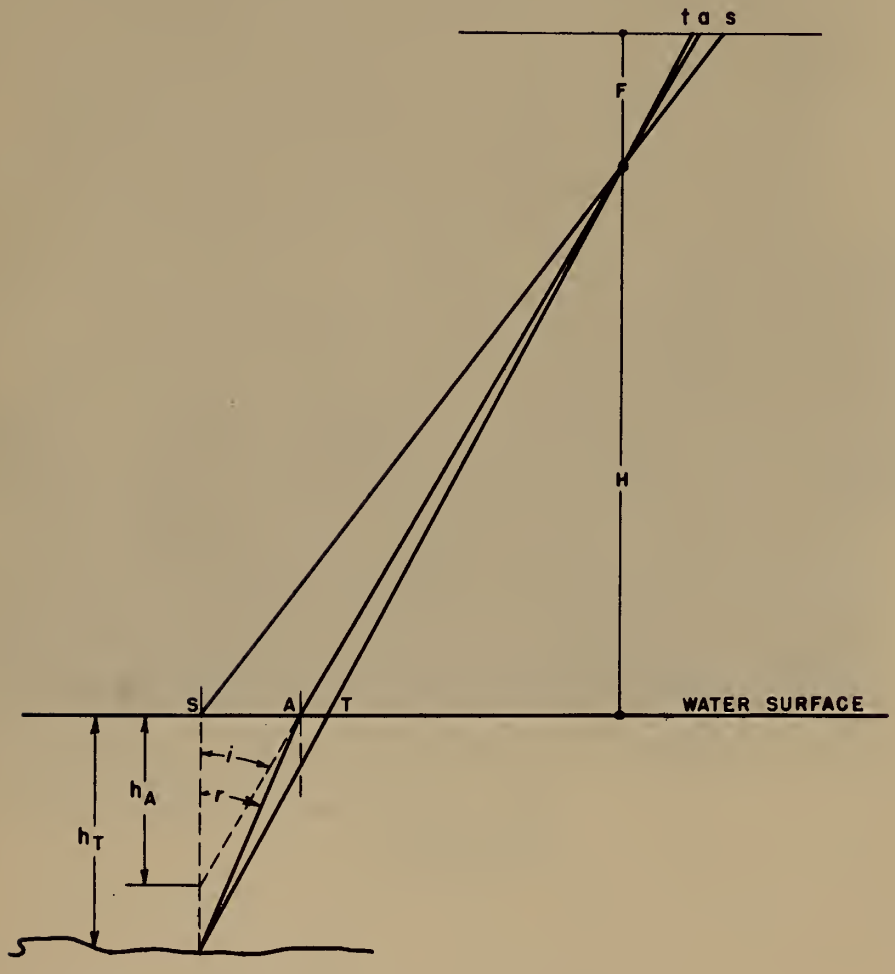



FIGURE VIC-52

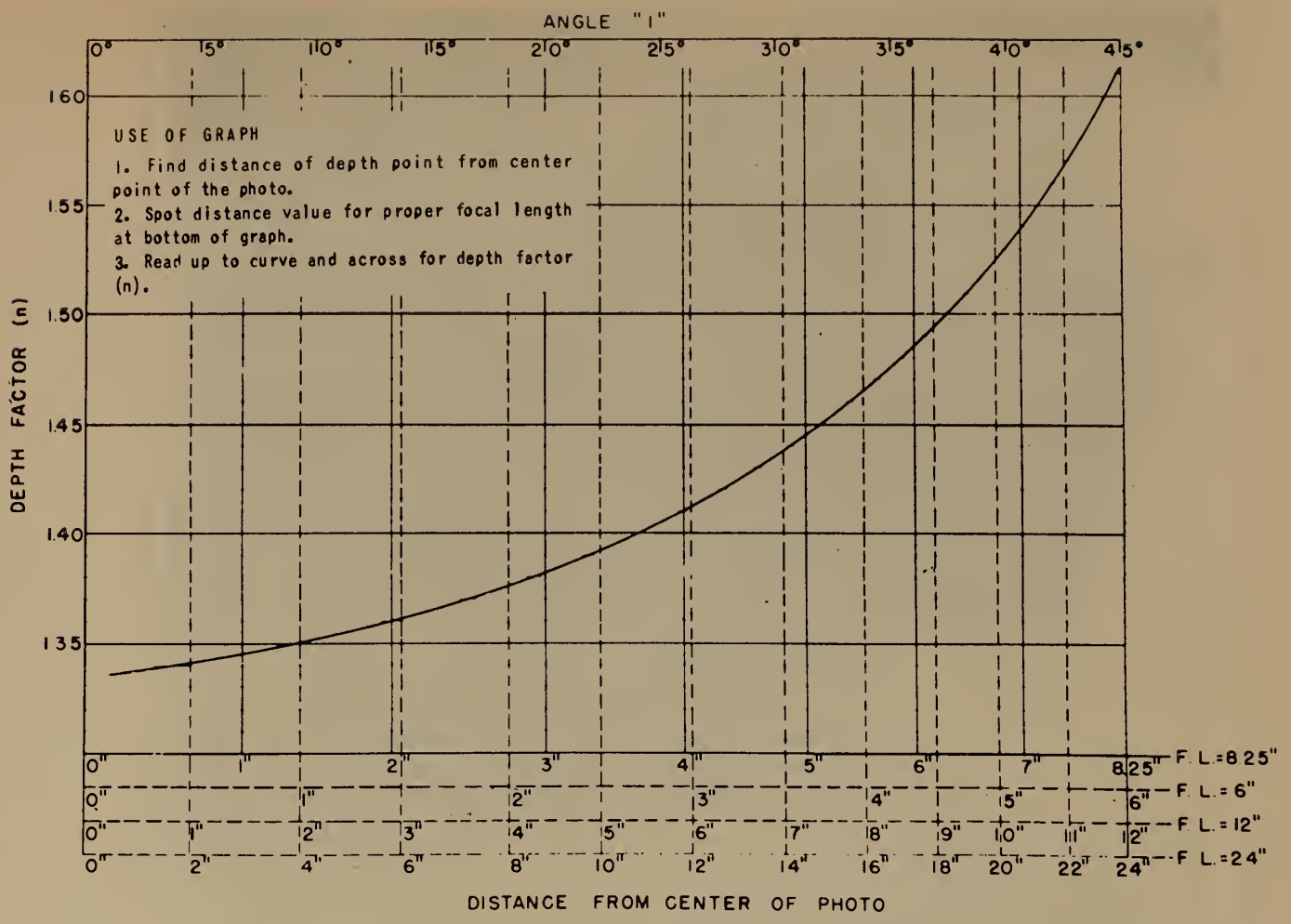



FIGURE VIC-53

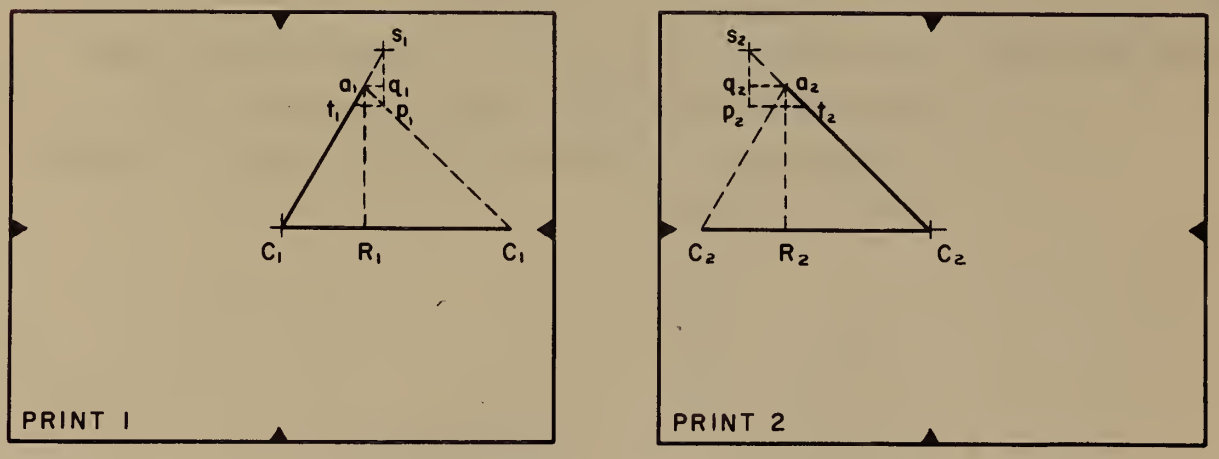



FIGURE VIC-54.

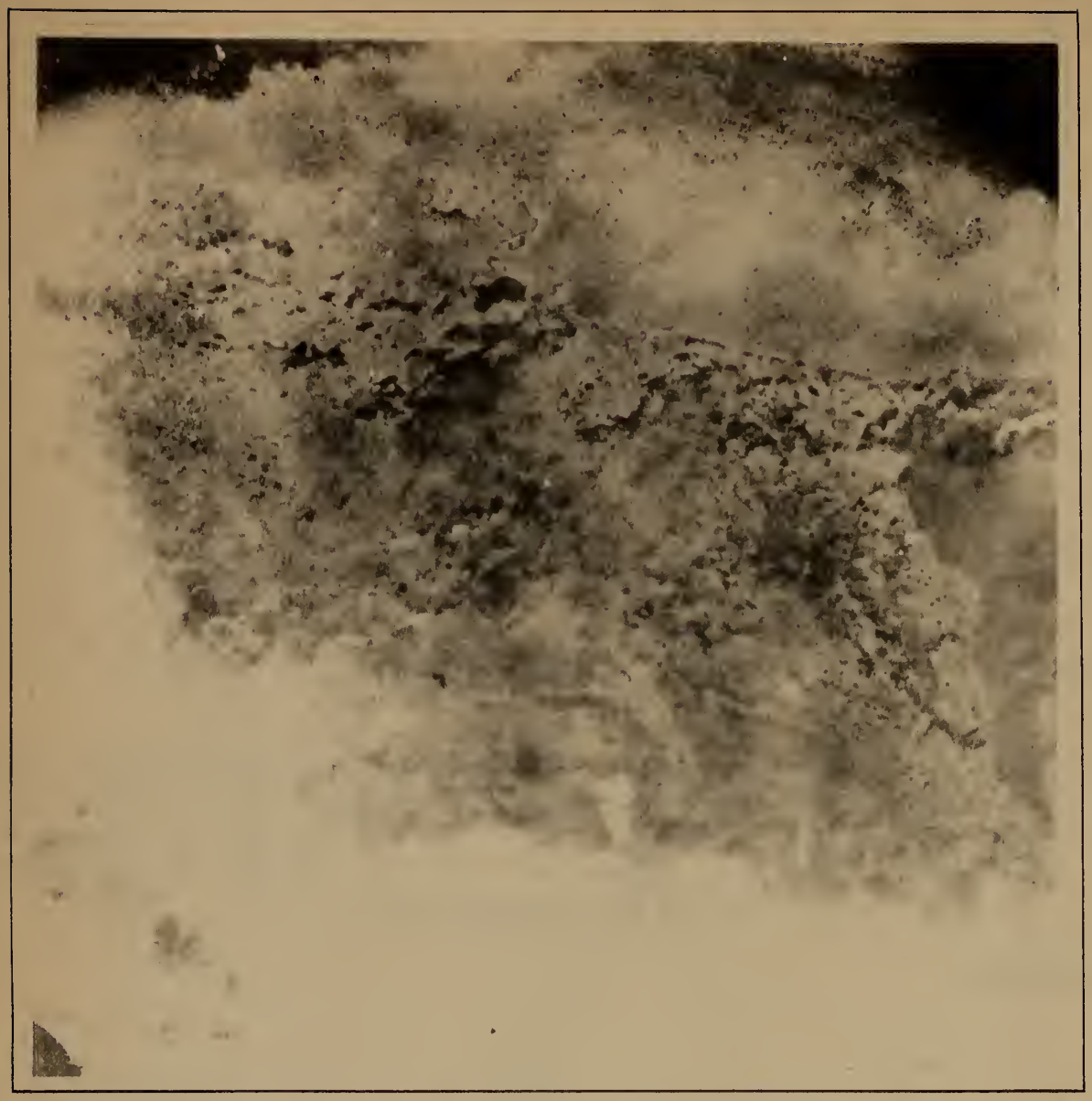


FIGURE VIC-55 - Experimental area at Cat Cay, Bahama Islands, where ideal conditions exist for bottom photography. The photographs were flown at 3000 feet using an F-56-8¼ inch F.L. camera.

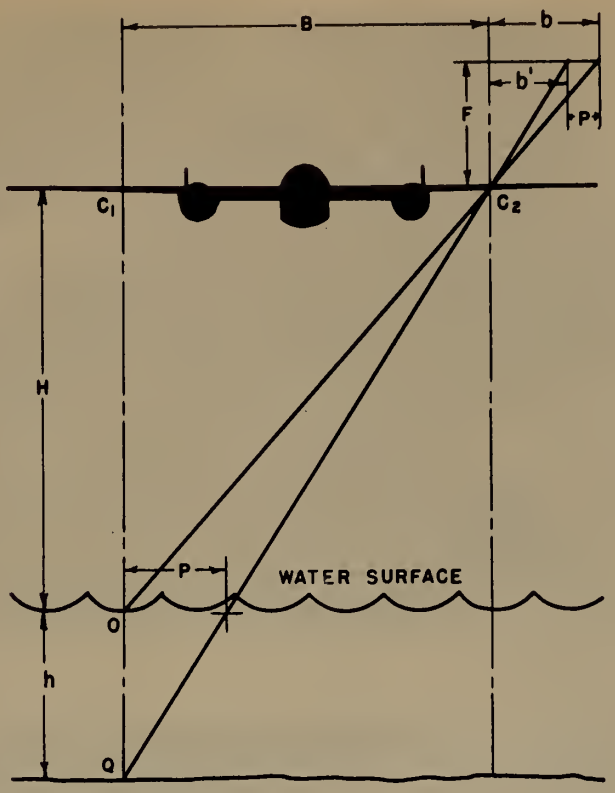



FIGURE VIC-56

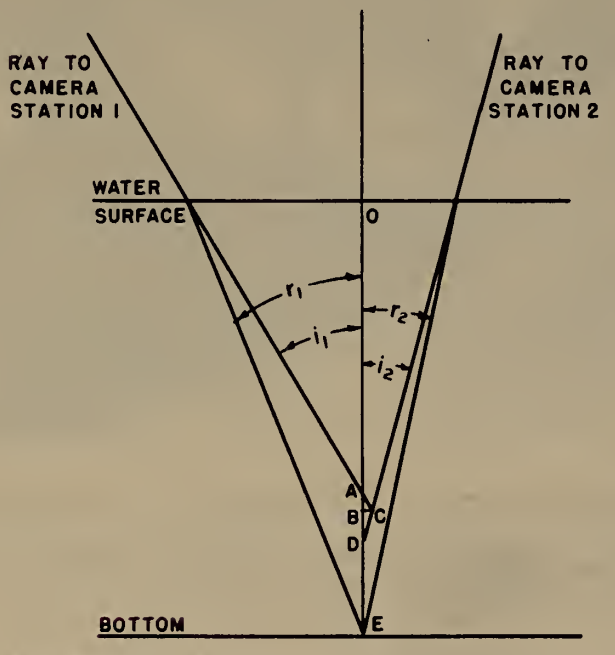



FIGURE VIC-57

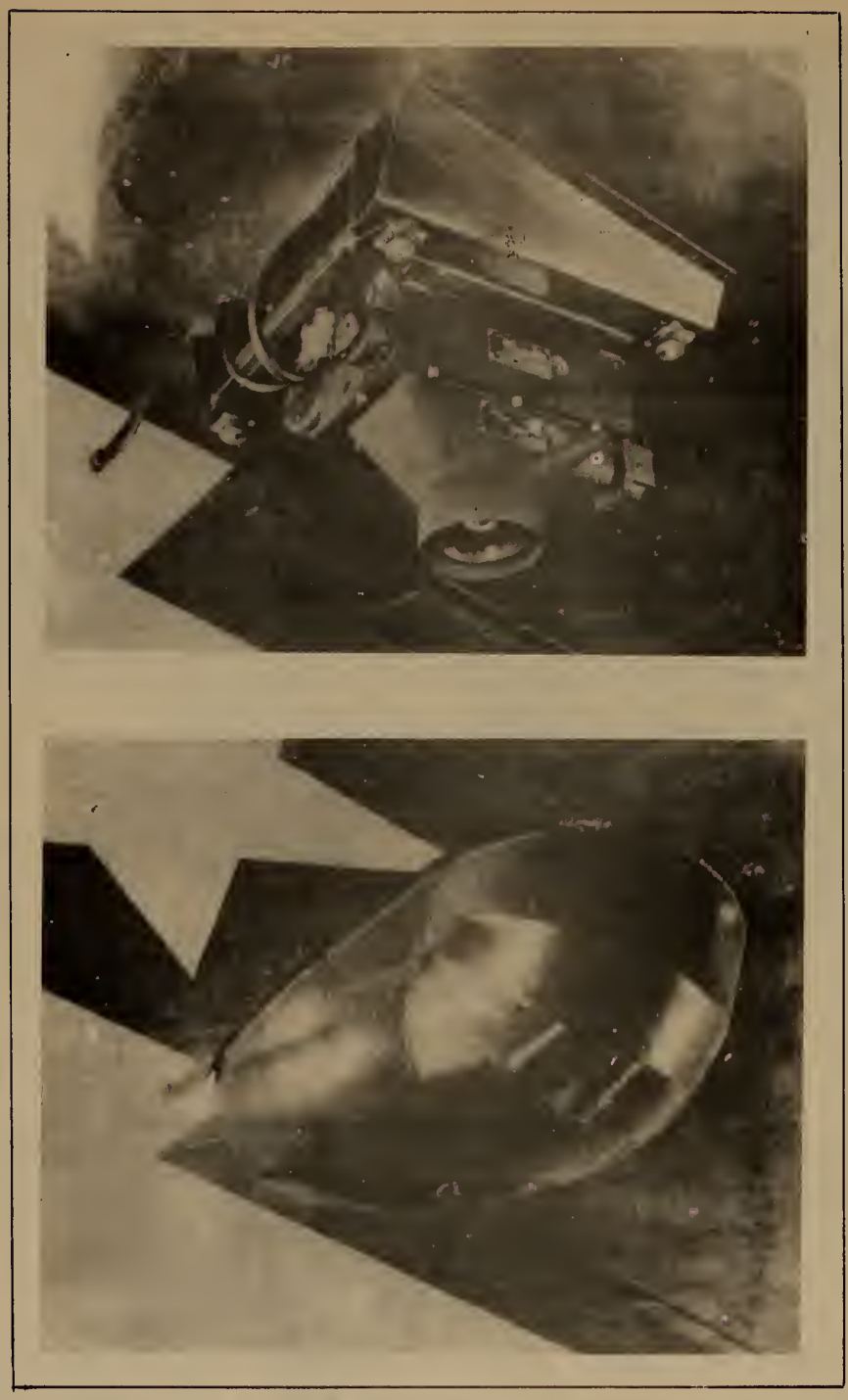
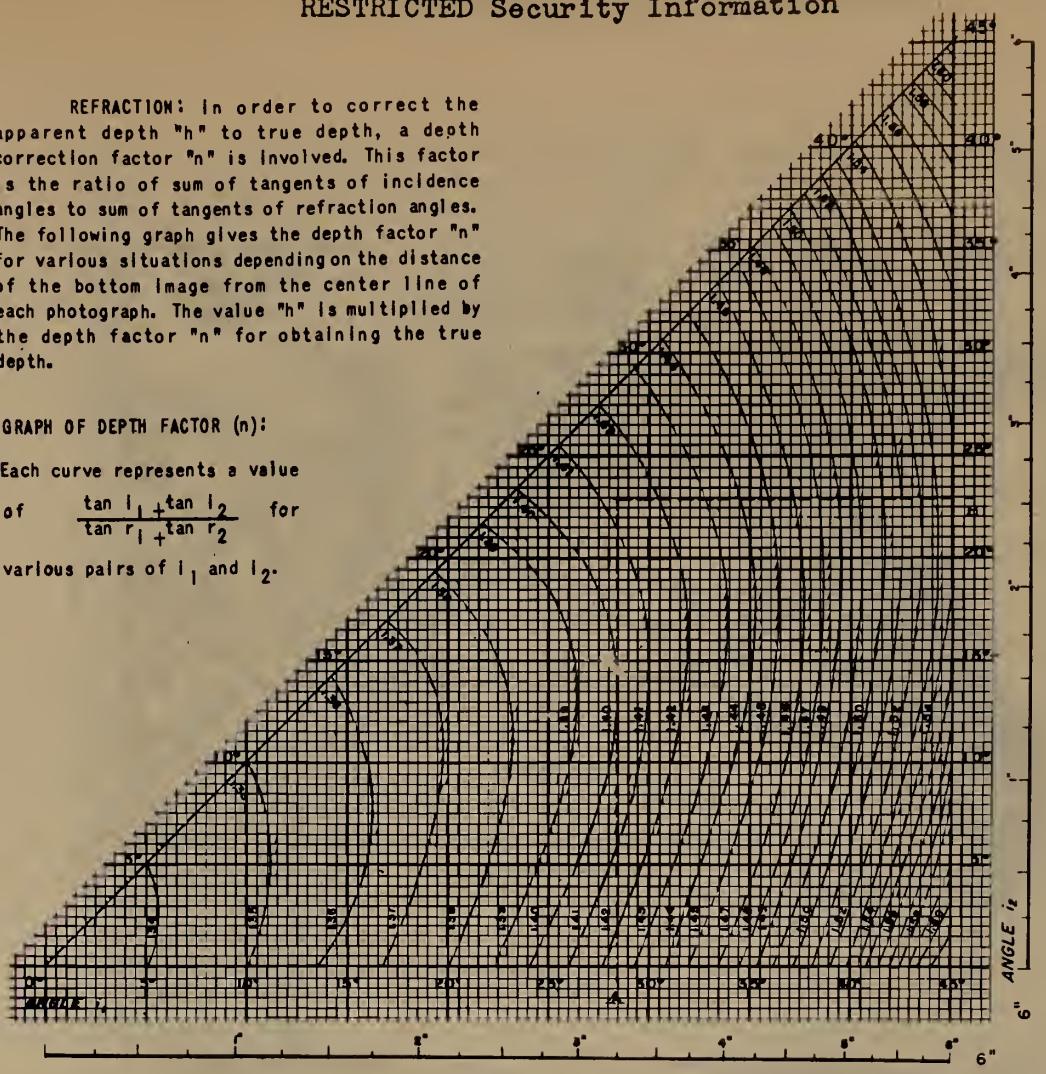


FIGURE VIC-58 - Photographs of the mock-up installation of a single lens Sonne camera in the wing of an F-6-F.

REFRACTION: In order to correct the apparent depth "h" to true depth, a depth correction factor "n" is involved. This factor is the ratio of sum of tangents of incidence angles to sum of tangents of refraction angles. The following graph gives the depth factor "n" for various situations depending on the distance of the bottom image from the center line of each photograph. The value "h" is multiplied by the depth factor "n" for obtaining the true depth.

GRAPH OF DEPTH FACTOR (n):

Each curve represents a value of $\frac{\tan i_1 + \tan i_2}{\tan r_1 + \tan r_2}$ for various pairs of $i_1$ and $i_2$.




EXAMPLE:
Point to be determined is 3.25" from the center line of one photostrip and 2.5" from the center line of the other. Focal Length 6".

The Correction Factor is determined by the point of intersection of a vertical line through "A" (Horizontal scale distance representing point 4.38" from the center line on one photostrip) and the horizontal line through "B" (Vertical scale distance representing point 2.5" from the center line on the other photostrip). The point of intersection is approximately three tenths of the way from the 1.41 curve to the 1.42 curve. The Correction Factor (n) then, is 1.413. Or TRUE DEPTH = APPARENT DEPTH x 1.413.

FIGURE VIC-59

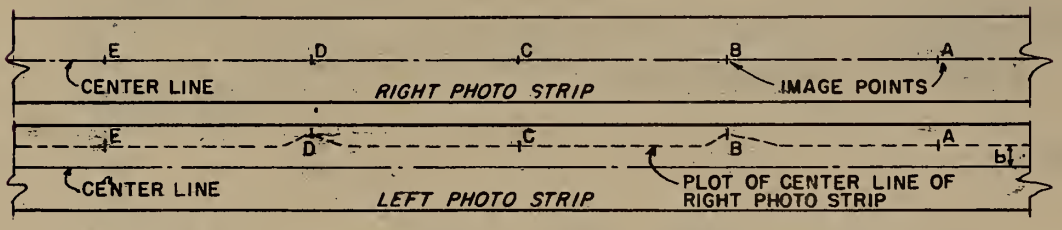
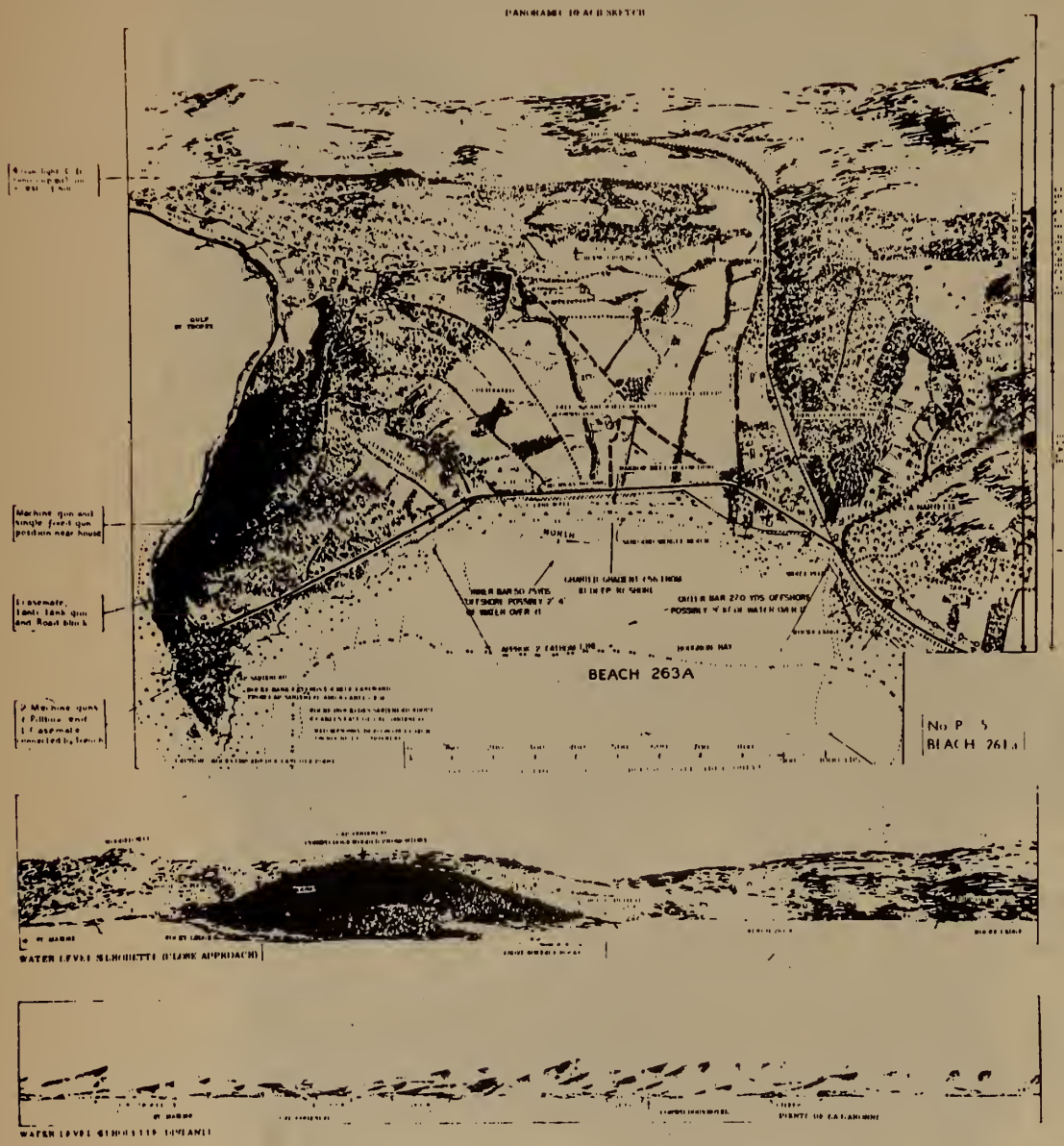



FIGURE VIC-60




Example of a beach sketch made for the invasion of southern France. The actual sketches used in the operation were twice this size and printed in 3 colors: Black for terrain, roads, underwater rock and fathom lines; Red for defenses and offshore sand bars and beach numbers; Blue for streams and ocean. Note how underwater beach gradients are shown, also the sand bar information given.

The water level silhouette helps craft identify the beach when approaching.

On the reverse side of each of these beach sketches was printed a 1:25,000 map of the area with defenses and hydrographic information overprinted in the same colors used for the beach sketch. A portion of one of these maps is shown on the next page.

FIGURE VIC-61

Portion of a 1:25,000 map with beach designations, defenses, and hydrographic information overprinted. Used for the invasion of southern France. See page 8.03 for full explaination of document.

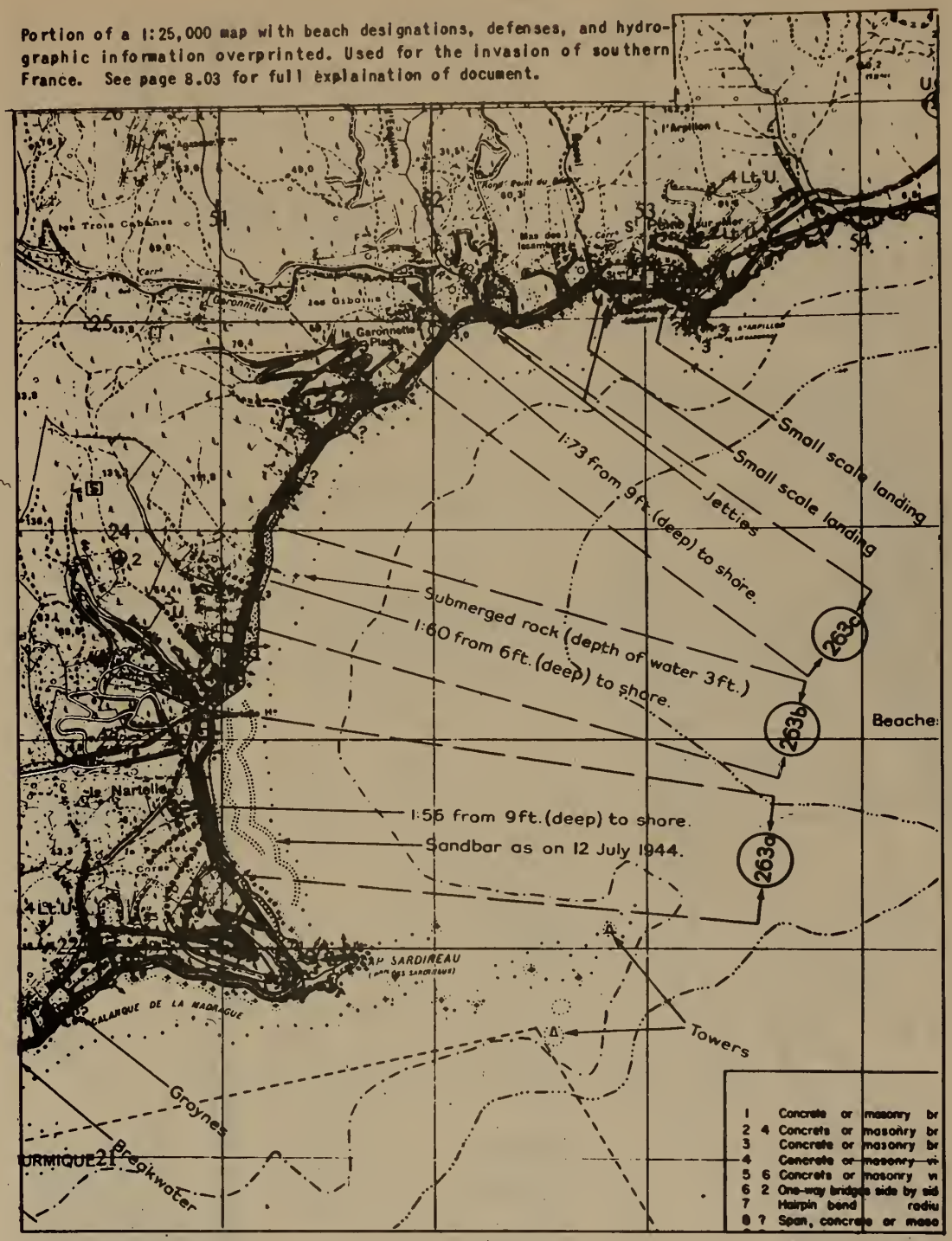



FIGURE VIC-62

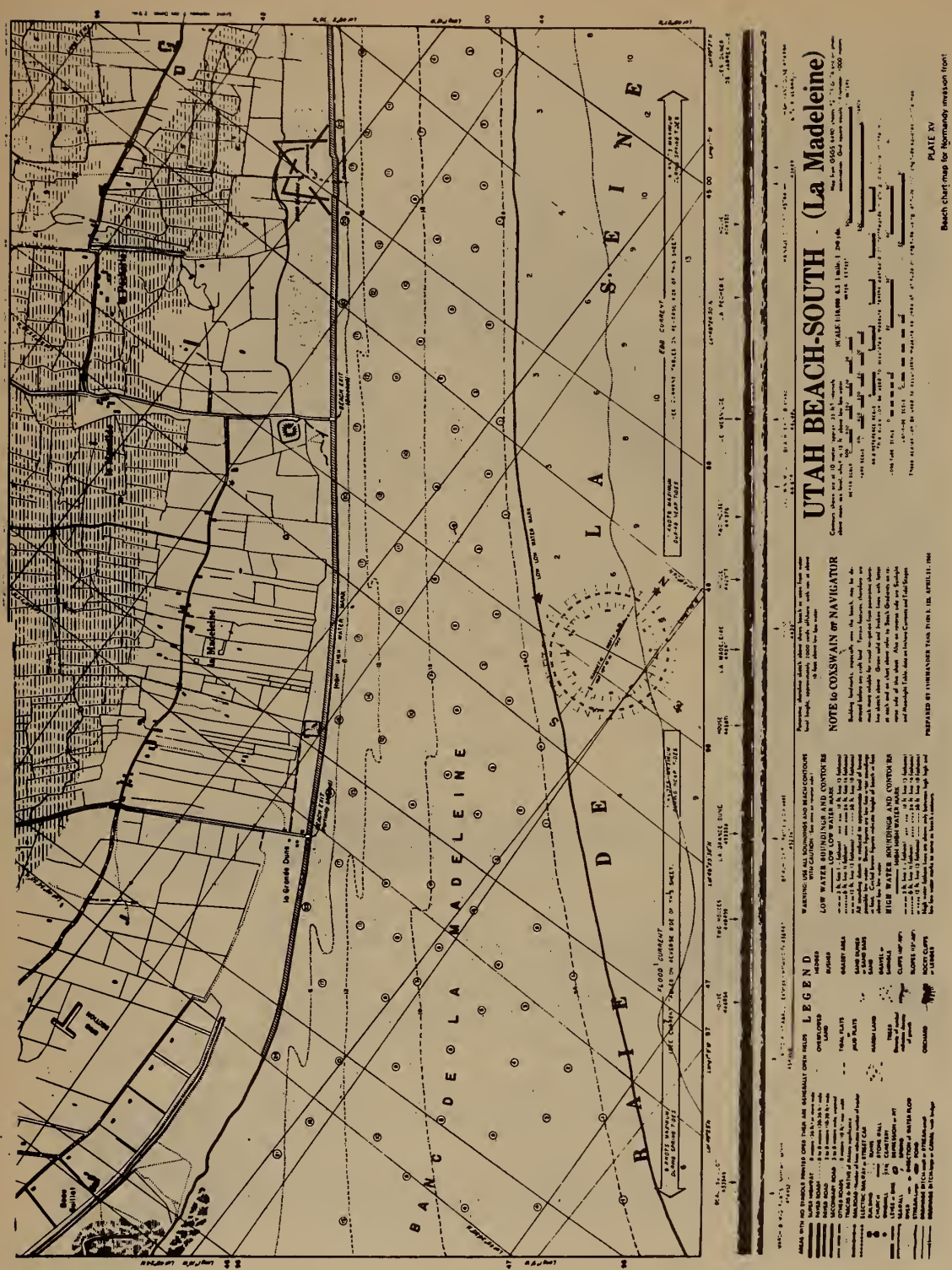



FIGURE VIC-63

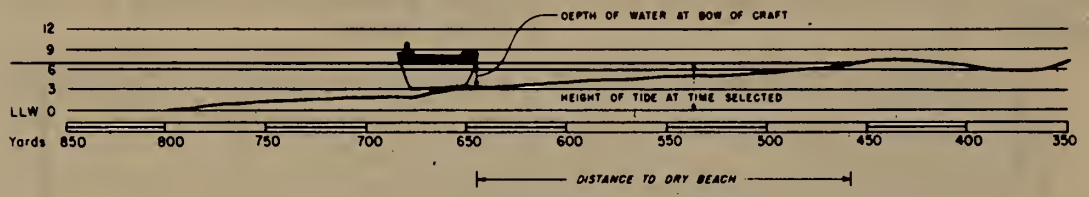
PREPARED BY COMMANDER TASK FORCE 122, APRIL, 1944

# LANDING CRAFT PROFILES

Profiles below are drawn to fit Beach Gradient Graphs on reverse side of either UTAH BEACH-NORTH or UTAH BEACH-SOUTH Shoreline Sketch. The long line running horizontally through each craft is the water line at normal loading. The profile of each craft below the water line conforms to the Beach Gradient horizontal and vertical scales. The distorted appearance is due to the fact that the vertical scale is 10 times larger than the horizontal scale, the vertical scale being 1 inch equals 22 feet 6 inches and the horizontal scale 1 inch equals 225 feet 0 inches (or 75 yards). The silhouette of each craft above the water line is drawn as it would normally appear without distortion to conform to enlarged vertical scale.

**HOW TO USE PROFILES:** (1) Determine height of tide from Tidal Stages Graph. (2) Find this height of tide on Beach Gradient graph (3) Place water line of craft at height of tide according to Beach Gradient vertical scale. Make sure craft water line is parallel to vertical scale lines. (4) Bring craft to right, inward land, until it touches ground. Distance from Beach Gradient graph line to craft water line is depth of water in feet at bow of craft. (read from Beach Gradient vertical scale). Read the distance in yards between bow of craft and dry beach from Beach Gradient horizontal scale.

**FOR EXAMPLE:** From the Tidal Stages graph we find that at the date and time we select the height of tide is 7 feet. We place the waterline of an LCT (5) approximately 7 feet above LLW on one of the Beach Gradient Graphs making sure the waterline is parallel to the 6 foot and 9 foot lines on the graph. We bring the craft to the right until it touches ground at about 650 yards from the back of the beach. Depth of water at the bow of the craft is about 3½ feet. Distance between bow of craft and dry beach is about 190 yards.



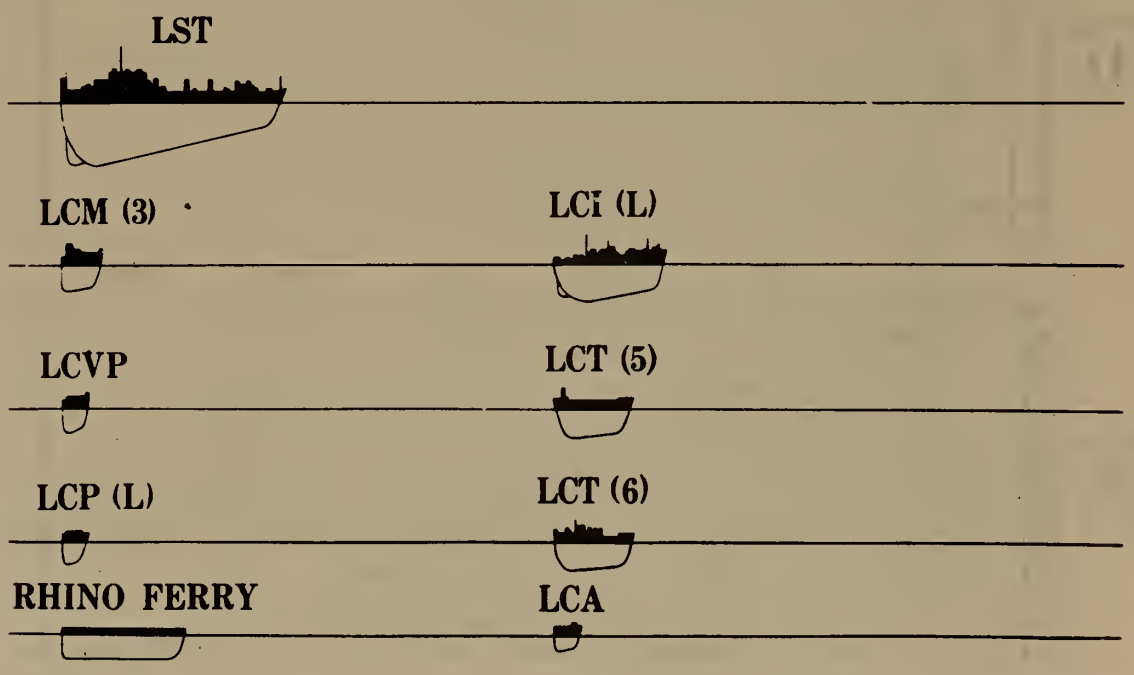




FIGURE VIC-64

## B. FINDING THE SUN ANGLE

The following must be known:

1. Month and day of photography.
2. Latitude and Longitude measured in degrees.
3. Angle of sun from horizontal required

(Adapted from the Nomogram developed by the National Defense Research Committee.)

PROCEDURE

1. Read D on above graph
2. Compute Lat.-D and Lat.+D. If either result turns out negative, the minus sign is disregarded.
3. Spot Lat.-D and Lat.+D on nomogram on opposite page; connect these points with a straight INDEX LINE (see sketch).
4. From the angle of sun desired, project left to INDEX LINE and downward to value of Local Sun Time.
5. Find Local Civil Time by adding or subtracting, as indicated, the value for the date of photography found in the time correction table.
6. Convert to G.C.T. or Local Zone Time as desired.

Time Correction (Equation of Time)

| | | | |
|---|---|---|---|
| 1 Jan. | +3 | 1 July | +4 |
| 15 Jan. | +9 | 15 July | +6 |
| 1 Feb. | +13 | 1 Aug. | +6 |
| 15 Feb. | +14 | 15 Aug. | +4 |
| 1 Mar. | +12 | 1 Sept. | 0 |
| 15 Mar. | +9 | 15 Sept. | −5 |
| 1 Apr. | +4 | 1 Oct. | −10 |
| 15 Apr. | 0 | 15 Oct. | −14 |
| 1 May | −3 | 1 Nov. | −16 |
| 15 May | −4 | 15 Nov. | −15 |
| 1 June | −2 | 1 Dec. | −11 |
| 15 June | 0 | 15 Dec. | −5 |

For daily correction see "Equation of Time" in Nautical Almanac.

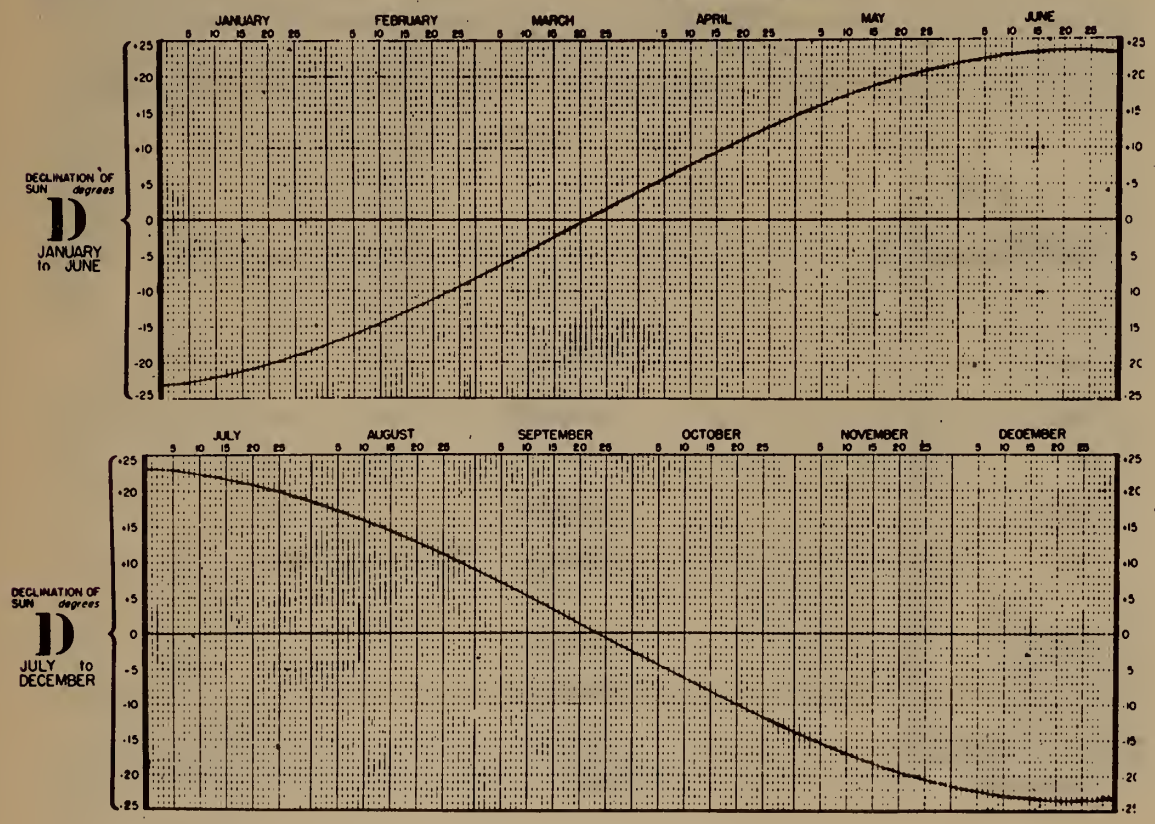



FIGURE VI C-65

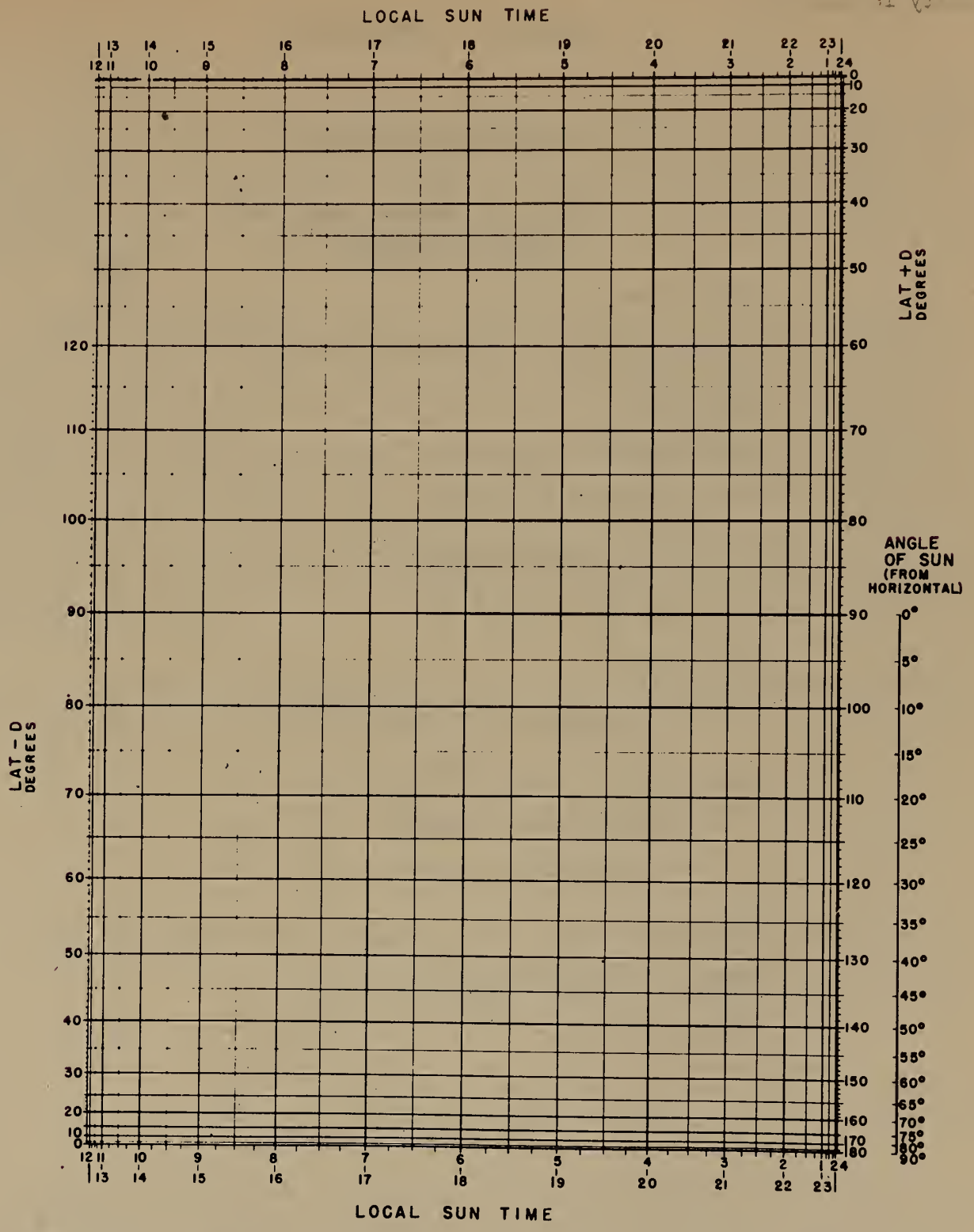



FIGURE VIC-66

MANUAL OF AMPHIBIOUS OCEANOGRAPHY

SECTION VI - INTELLIGENCE

## D. AMPHIBIOUS SURVEYING

BY
LT. D. A. PATRICK, CEC, USN

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# D. AMPHIBIOUS SURVEYING

## 1. Introduction:

In surveying for amphibious operations, the main requirement is to find the quickest, most practicable method of obtaining information to plot beach and surf surveys to the required accuracy. There is considerable literature on hydrographic surveying in general, but not much attention has been paid to surveying in the surf, nor to many of the details of operation. With the recent rise to prominence of amphibious operations, the need has developed for a special type of surveying - namely amphibious surveying.

Amphibious Surveying is merely surveying which includes both hydrographic and topographic portions, and the transition between them. For amphibious military operations, these surveys are much more detailed than normal hydrographic surveys, and include: measurement of profiles perpendicular to the beach, and the charting of bars, reefs, beach forms, obstacles, anchorages, etc. The most critical part of an amphibious survey is in the region extending from a depth of 3 fathoms below tidal datum offshore to the line of permanent vegetation on shore. Therefore, this area must be surveyed carefully. Best coverage is obtained by surveying the water portion at high tide, and the beach face at low tide: thus getting an overlap of data. In general, depths to the nearest foot are satisfactory for the offshore portions, and elevations to the nearest one-half foot are desirable on the beach. Horizontal control to the nearest 10 feet is desirable.

In an amphibious survey, there are normally three fundamental variables. (These in turn depend on the extent, use, time, equipment and personnel available; and on the required accuracy of the survey):

(i) Methods and Equipment (in Field Work)

(ii) Control Methods (in Field Work)

(iii) Office Work

Plan and Limits of this report: The section is divided into three main parts:

(i) Special problems of amphibious surveying - including surveying in the surf.

(ii) Field Work - including Methods and Equipment, and Control

(iii) Office Work - including reduction of notes and plotting the finished surveys.

This study is limited to measuring and locating depths of bottoms, and does not cover the recognizing, locating, and plotting of obstructions and defenses. Certain phases of amphibious surveying are described in detail because these methods are known to be satisfactory. However, actual paractice will doubtless vary with individual surveyors.

## 2. Special Problems of Amphibious Surveying:

Amphibious surveying presents many special problems not usually associated with other types of surveying. The best possible information must be obtained, often under adverse conditions of weather and enemy action; with meager amounts

of personnel and equipment; or without adequate control. Nevertheless any information, meager or not, may be of great value if the reliability is known.

Seaward of foam lines, depths are subject to a correction for wave height. In deep water, where the depth is greater than half the wave length, the trough and crest are about equi-distance below and above the still water level, (See Figure VI D-1). However, as the wave passes into shallower water, the crests peak up so that the trough comes closer to the water level. In the area from about 80 foot depth to the breaker line, the still water level is taken as one-third of the wave height above the trough. This depth of 80 feet is taken for waves of 12 seconds-period with the transition point for the change in correction based on $d/L_o = 0.11$. Thus if the wave period is much different from 12 seconds, this transition depth can be estimated from the relationship

$$80\ T^2/(12)^2$$

where T is the characteristic wave period. Thus for waves of 6 seconds-period the transition depth is about 20 feet and for waves of 18 seconds-period the transition depth is about 180 feet. These illustrated relationships are a practical compromise of a combination of many complex factors such as variations in beach slope, wave steepness, wave period, and possible other factors. As they are based on laboratory data obtained from idealized wave channel experiments, refinements are not warranted. Validity of these relationships decreases as wave heights and variabilities increase since some of the variations mentioned above will deviate more from the norm. These ratios have been found to be reasonably reliable for wave heights of up to about 6 feet. Although soundings can be taken in higher waves, the validity of the survey becomes more doubtful.

If a fathometer is used, the record may be corrected by drawing a line through the indicated waves on the record at one-third the distance of the trough to the crest (Figure VI D-2). Note that the trough and crest are inverted on the record, i.e. the crest is closer to the bottom of the record. If a lead line is used, the sounding is taken and recorded in the trough. Then the height of the next wave is estimated and recorded. When the notes are later reduced, one-third or one-half of the wave height (as appropriate) is added to the sounding. Experienced personnel may apply this correction as the sounding is taken, but the additional possibilities of error should not be overlooked.

Correction is not considered necessary in areas where waves are advancing as foam lines if soundings are taken in the trough since (as shown in Figure VI D-1), the trough and still water level have been found to roughly coincide. Corrections to fathometer records inside the surf zone are of no concern since suitable records cannot be obtained because of turbulence of the water.

Another phenomenon which introduces errors in soundings, in and near the surf zone, has been described as "surf beats" by Munk (1949) (VI D-10).* This phenomenon has been observed by Munk and others (Bascom, 1950a) (VI D-2) on the Pacific Coast of the U. S. and presumably occurs on other shores of the world as well. The time interval of these surf beats on the U. S. Pacific Coast is of the order of $2\frac{1}{2}$ or $3\frac{1}{2}$ minutes and is manifested by an increase in the violence of the breakers accompanied by a general rise of the water surface, usually from a few inches to a foot or more

* Hereafter, author's last name, followed by the date of publication (in parentheses) refers to the "References Cited".

when the water piles up onshore. Munk has obtained a limited correlation between the surf beats as recorded by a special long-period (tsunami) recorder off Scripp Pier at La Jolla, California, by assuming the seaward transport of water in the surf zone to be constant and by integrating the swell record over a period of about twenty minutes (Figure VI D-3). From a practical standpoint, however, the effect of surf beats on the accuracy of hydrographic surveys in the surf zone is usually neglected. If possible the presence or absence of surf beats and the approximate amplitude should be noted in order that the reliability of the survey in the surf zone may be determined. Random observations of changes in trough ele vation over a period of a few minutes on a temporary staff in the surf close to shore may be sufficient to describe the surf beats.

After the survey has progressed from offshore through the surf zone, the question arises: Where does the hydrographic portion of the survey end? i.e. Where is the still water line on the beach? (See Figure VI D-4). Bagnold (1940) (VI D-1) conducted a series of model tests on the movement of sand on and off beaches resulting from wave action. He noted that with waves of uniform period and height on a beach of uniform slope, the still water level corresponded very closely with the limit to which each breaker retreated seaward before being over- taken by the succeeding breaker. This limit is defined as the "limit of backrush and is characterized on ocean beaches by a low muddy swirl. Correspondingly, the upper limit on the beach face to which the breakers advance has been termed the "limit of uprush". Actually, on ocean beaches wave periods and heights are irreg ular and beach slopes are far from uniform. Some limited field observations have been reported (Patrick, 1950) (VI D-11) in which relationships were examined be- tween the limit of backrush, limit of uprush and still water level on some Pacif Ocean beaches. Some 150 individual random measurements of elevation of limits of backrush and some 100 measurements of limits of uprush were recorded.

These limits of uprush and backrush would be expected to vary in elevation for different single wave energies, for different group wave energies, for differ ent slopes of beach face, and possibly for other factors. The above measurements of elevation were then compared with the U. S. Coast and Geodetic Survey's tide predictions, by reference to local third order bench marks which had been tied into the level net of the U. S. A more significant comparison between these limits and the tide stage could have been made if a suitable tide gage had been available. However, tide records and bench marks are not found for most beaches of the world. Consequently, the object of the studies was to determine whether or not a reasonably reliable correlation could be obtained between backrush or uprush and the predicted tides which are published for a large number of ports throughout the world.

The results of the study have been plotted in Figures VI D-5 and VI D-6. No reliable relationships were noted, although the best correlation was noted between the limit of backrush and the tide predictions. When the tide pre- dictions were rounded off to the nearest foot and the corresponding elevations of backrush for each one-foot increment of tide predictions were averaged, the elevations of these limits of backrush agreed within one-half foot of predicted tide stage (Figure VI D-5). However, in view of the variability of the in- dividual elevations included in the averages, this apparent agreement is question- able until confirmed by more measurements. These data were manipulated further, but only the following rather negative conclusions resulted:

(i) No simple empirical relationships exist between elevations of limits of backrush and uprush, and those given in the United States Coastal and Geodetics Survey tide predictions.

(ii) The limit of backrush is a more reliable index to tide prediction than the limit of uprush, and is recommended as the point for the "zero" sounding.

(iii) The variabilities of the elevations of limits of backrush and uprush do not appear to be a function of the tide stage.

(iv) The variabilities of the elevations of limits of backrush and uprush are at best very obscure function of the beach slope.

The Waves Investigations Group at the University of California has attempted to relate empirically the still water ocean level (i.e. the tide stage) to the ground-water level behind the beach. Although unsuccessful, the results of this study have been reported by Isaacs & Bascom (1949)(VI D-9). The report presents ground profiles, water table profiles and specific gravity measurements (as a salinity index). Although the water table profile fluctuated somewhat with the tide stage, no correlation between the two could be established.

Briefly, then, the phenomena which should be taken into account for surveys in and near the surf zone are the relationships between the wave heights and the still water level, the surf beats, and the determination of the still water level on the beach.

3. Field Work:

a. Method and Equipment:

Various methods and equipment may be used for an amphibious survey; restrictions and expendiency will usually dictate the final choice. The three most common methods of hydrographic surveying are by lead line, fathometer, or sea sled. Discussed below are methods which have been reported reasonably successful under various conditions:

(1) Lead Line Method: This method of soundings has two advantages: special equipment is kept to a minimum and required equipment is easy to maintain.

An operation report (Fielding 1950) (VI D-7) describes the Amphibious Surveying methods of Underwater Demolition Teams (UDT):

"To review operating procedures, swimmers are dropped, in pairs, from a LCPR running parallel to the beach. This distance between pairs is 100 yards. Each pair carries as equipment a 30-foot lead-line marked in two feet intervals and a slate upon which the information is written. Swimmers sound for predetermined depths (i.e.--depths within the three fathom curve compensated for tidal stage) and estimate the distance offshore at which these soundings are located. This form of reconnaissance is known as a swimmer or string recon. Another form of reconnaissance employed by UDT is the flutter board or line reconnaissance - swimmers are assisted by a 500 yard reel of line marked in 25 yard intervals which is staked into a range upon the beach. In this form of survey, depths are recorded every 25 yards and compensation for tidal stage and wave height applied following completion of the reconnaissance. Normally, charts and aerial photographs are made available to UDT prior to scheduled operations and swimmers can

be briefed as to approximately where the 3 fathom curve will be found, width of beach, height of scarp, exits, etc... ."

When a small boat or DUKW is used for the surveying, methods vary somewhat for taking lead line soundings. The lead line is usually made up by the surveyors. About 60 feet of line should be provided although only about the first 35 feet is normally used. The additional length is handy for individual soundings in deeper water, as when setting a wave recorder. All lines should be checked periodically for signs of wear and then re-calibrated.

A three or four-pound lead is usually sufficient for hand soundings. A 25 pound lead has been used by the Beach Erosion Board (Bulletin, 1947) (VI D-5). This lead was secured to a line which operated from a hand winch through a davit, hung over the side of a DUKW. This lead was used for surveying in deeper water where the lead was not pulled up out of the water after each sounding. Thus a heavier lead was required in order to minimize bowing of the sounding cable. A much lighter lead, which can be carried by a swimmer, is used by UDTs.

The lead line is apt to give erroneous readings for the following reasons: The line is not vertical at the time of the reading, the line is bowed, the reading is not taken in the trough, and the lead may sink into the bottom (if it is muddy).

When taking a sounding with the lead line by hand from a DUKW or small boat, the line is thrown ahead into the crest of the wave. As the lead sinks, the craft advances so that the line is vertical when the lead grounds. As the leadsman "feels" bottom with the lead he reports the reading in the trough of the wave. As he retrieves the lead he reports the approximate height of the following waves. (Hand lead soundings from a small boat are illustrated in Figure VI D-8.) Such soundings taken when waves are of the order of 4 feet high are believed to be accurate plus or minus one foot (neglecting surf beats in the surf zone). Figure VI D-9 shows a repeat run of soundings over a range with relatively inexperienced leadsmen. The surf zone is the area where accurate information is desired most for an amphibious operation, but unfortunately chances for errors are greatest there. On steeper beaches, the surf zone can often be more accurately surveyed if the hydrographic portion is surveyed at high tide and the topographic portion at low tide. Thus the data will overlap and a check on the work is provided. On flatter beaches, however, the surf zone is often wide so that much data must be taken in the surf in spite of the overlap.

(2) Fathometer Method: Fathometers are of particular advantage when extensive surveys over an area are required. They are normally carried by a DUKW or small craft. The Beach Erosion Board (1947) (VI D-5) has found the fathometers to be superior to lead line and twice as fast on some hydrographic surveys at Pensacola, Florida and in the Gulf of Mexico. However, their soundings ranges varied from 2,000 feet to 17,000 feet long. The ranges were sounded under conditions varying from negligible to strong littoral current, calm to windy weather, and from practically still water to about three-foot waves. Their normal operation speed for the DUKW when sounding was 4 to 5 miles per hour. They used at least three bar checks daily with the echo sounder. (A bar check is made by lowering a steel bar to a known depth in the approximate working range of the soundings. The fathometer record is then adjusted to record the correct depth to the bar.)

Caldwell (1950) (VI D-6) has written an interesting and informative article on fathometers. He discusses elements of operation, various basic types, inherent limitations, and common discrepancies associated with echo sounders. The applications which he discusses are broader in scope than amphibious surveying on ocean beaches.

RESTRICTED Security Information

The fathometer record is subject to errors caused by the following varying velocities of sound in water (due to varying salinities and temperatures); mechanical and electronic errors in the equipment; linear interpolation between position fixes (a large number of fixes reduces this error); and motion of the craft (pitching and rolling) which may exaggerate wave heights.

For short periods of intermittent surveying, the disadvantages of the fathometer compared to lead line soundings will generally outweigh the advantages. Figure VI D-7 shows a fathometer (NK2 Bludworth) installed in a DUKW. Compared with lead line soundings the following advantages and disadvantages of the fathometer have been noted:

Advantages:

(i) A more detailed record of bottom is obtained; that is, a continuous record is obtained as opposed to intermittent lead line points. Thus, even small bars are apparent.

(ii) There is less work for the operator and the mass of the carrying craft averages out small surface irregularities.

(iii) Tide can be calculated at the time and set off as an adjustment, so that the tide correction is included on the field record.

(iv) A permanent record of the profile is obtained immediately.

(v) Fathometers will survey to a greater depth than is practicable for lead lines. (However, amphibious surveys usually end at about thirty foot depth.)

Disadvantages:

(i) Turbulent water (breakers or foam lines) produces a blank record.

(ii) It is difficult to interpret records taken over a mud bottom.

(iii) Waves show in the records as opposed to lead line soundings made in the troughs and corrected for actual wave height. Pitching motions of the craft exaggerate the wave heights recorded by the fathometer so that errors might occur when the profile is corrected later in the office.

(iv) Electronic and mechanical gear is subject to failure when used in proximity to salt water. Not practical to house it in tight compartment.

(v) As used on DUKWs, the sending-receiving unit is outboard and must be dismantled at the close of a day's work.

(vi) Voltage required is 12 volts (not convenient on a DUKW).



RESTRICTED Security Information

(vii) None of the other work on the beach is changed. Just as much surveying equipment and effort is required.

(3) Sea Sled Method: Perhaps the most accurate method for obtaining profiles through the surf zone is to use a sea sled with a graduated mast. (See Figure VI D-10). If the sea sled elevations are recorded by transit-stadia method, only one station ashore need be manned since elevation, stadia distance and bearing can be recorded by one instrument. The Waves Project (Isaacs, 1945) (VI D-8) has reported successful surveys using a self-propelled sea sled from the shore to depths of about 20 feet. The accuracy of elevations reported was to the nearest 0.1 foot, and the accuracy of horizontal distances was plus or minus 1%. In this method, a 22-foot mast was placed on the sea sled and was graduated in feet and tenths.

On Naval Research Project NR 257-201 (Amphibious Oceanography), sea sleds with masts graduated in feet were used so that elevations to the nearest half foot could be read easily. Positions were located by a two instrument fix. These sea sleds were not self-propelled, but were towed to sea by a DUKW (with a retrieving line retained on shore).

On one such survey (Figure VI D-11), positions were fixed at approximately 75 foot intervals as the sea sleds were pulled ashore. This method was not employed extensively inasmuch as a single sea sled, properly rigged was not available because of other higher priority investigations. Self propelled sea sleds in the larger sizes required to support the mast did not prove too successful, although all possibilities were not exhausted because of the higher priority of other oceanographic investigations on the field trip. Presumably, however, a suitable sea sled arrangement (either self-propelled or towed to sea and retrieved by a hand winch) could be designed for surveying the surf zone. Such an arrangement is, of course, independent of tide stage and surf beats. It is accurate to one-half foot for reasonable slopes. A self-propelled arrangement would be especially desirable, because it would permit a survey in surf conditions hazardous to craft.

b. Control:

Methods of control to fix locations and orient the survey will vary according to the accuracy required, and the equipment and facilities available. Normally, the survey is best controlled from shore, although the standard three point fix method (described by Bowditch) (1943) (VI D-4) with two sextants can be used if there are sufficient land marks ashore. For limited hydrographic surveys, the use of shoran or other electronic position locating devices will not usually be justified, due largely to the fact that the various land and sea stations are usually intervisible. i.e. all other stations can be seen from a given station. Normally, then, the control is maintained using standard surveying instruments or human judgment.

(1) Control Methods Used by UDTs: As noted in the discussion on lead lines, UDTs use two methods for control of their surveys: estimating position and the line reconnaissance. Both are simple and direct. The accuracy of the estimating method depends largely on the training and experience of the estimator. Distances over water are especially difficult to estimate, though some persons can develop remarkably accurate judgment.

The line reconnaissance is limited in extent by the 500 yard reel of line. In regions of high surf or strong littoral currents, the swimmers may not be able to maintain range or carry the line out to sea.

(2) Two-Transit Method: The two-transit method may be used for hydrographic survey control when the shore may be occupied, when radios may be used and when range lines for the profiles either have not been established or the sounding craft cannot stay closely on range because of littoral currents or other factors. When this method is used, several adjacent profile ranges may be run without changing instrument stations ashore. All notes are recorded ashore. Figure VI D-12 illustrates three profiles taken by this method.

The Beach Erosion Board (1947) (VI D-5) used a slightly different two-transit method. One transit establishes the range line and keeps the sounding craft from one transit to the other is always greater than 30°. For both these variations of method, radio communication is maintained between the two shore stations and the sounding craft. Normally, when using DUKW's this method requires a seven man crew as follows: two instrument men and two recorders ashore; one driver, one radioman (assistant driver) and one leadsman at sea. On routine surveys, a standby vehicle is usually employed and standby driver and assistant may also be required if the instrument man and recorder at one of the shore stations cannot operate the standby vehicle.

(3) One-Transit Method: This method is recommended under conditions similar to those above if the sounding craft can satisfactorily stay on range; if only one profile in the vicinity will be taken; or if the weather is too inclement for the plane table method described below. The one-transit method is very similar in principle to the two-transit method above, except that the sounding craft is presumed to stay on range. Thus, an instrument man, a recorder, and one of the transits are all eliminated.

As an example, when the one-transit method is used in conjunction with lead line sounding, the following equipment and procedure are recommended:

Equipment consists of:

Transit
leadline graduated in feet
stadia board
pair of hip boots (these are used in cool water when no amphibious vehicle is available for the beach survey) (i.e. on days when only the upper beach is surveyed).
two portable radios (Model SCR 300 have been used)
DUKW for soundings
one standby vehicle for safety (DUKW or LVT)

Procedure for water sounding:

Transit is set up and backsighted along the base line.

DUKW proceeds seaward to about thirty feet of water, and is oriented on range by the driver (by aligning fore and aft range poles on shore)

DUKW then proceeds to shore on this range.

Lead man takes soundings by throwing the lead into the crest of a wave as it passes the DUKW, and reads the sounding in the trough. He notes the height of the following wave above this trough and then reports this sounding and the wave height to the radio operator.

Radio Operator radios this information to the recorder on shore

Instrument man, meanwhile, has sighted the lead as it was thrown, He now clamps the transit, reads the angle, and reports the reading to the recorder.

Soundings continue thus at the rate of about two per minute until the DUKW is no longer waterborne.

The lead line is then laid aside, and the stadia board is used to measure depths directly.

The final sounding is taken at the limit of backrush - which is considered to be the tide stage at the time.
The recorder notes the time that a run of soundings starts, and how long it lasts. The mean of these two times is later used to compute the tide stage.

When the one-transit method is used with the fathometer, precalculated angles may be set on the transit which give a point of intersection of the range in even hundreds of feet from the beach.

As the DUKW moves in on the range, the radioman at the transit calls out the number of feet as the DUKW passes the point of intersection.

The fathometer operator makes a "pip" on the chart and writes in the distance.

The transit man then may set the next smaller angle on the transit, and when the DUKW appears in the crosshairs, he repeats the process.

The illustrated typical chart (Figure VI D-2) shows the results that may be expected. Other pertinent information can readily be written on the chart when necessary. These records would normally be made at high tide; in the turbulent zone they overlap stadia surveys made at low tide if the surf zone is narrow enough.

A comparison of the fathometer and lead-line sounding methods was made at Carmel and the results for one range are shown in Figure VI D-13.

Beach land profile data are obtained by setting up the transit on each range at some known distance from the base line. Significant elevations are taken by the transit-stadia method.

(4) Plane-Table Method: This method requires the same survey party as does the one-transit method, but uses different equipment and a different method of control. An alidade and plane table are substituted for the transit.

For the water soundings, the instrument man lays out a transparent plane table sheet which shows the profile ranges and base line, preferably to the same horizontal scale as will be used on the finished profile sheets. A typical field plane table sheet is shown in Figure VI D-14.

The same instrument station is used as for the transit method.

The instrument man then orients the plane table.

The leadsman in the DUKW takes soundings and reports (as described for the transit method) at the rate of about three or four per minute.

As the lead line is thrown, the instrument man sights it, and scribes the line of sight on the plane table sheet at the intersection with the sounding range.

The recorder then notes the consecutive number and the depth of the sounding at the end of this line of sight on the plane table sheet. (Additional profile runs may be controlled from the same instrument station if they are a reasonable distance from the instrument - say between 500 and 1500 feet).

The base line should be back-sighted after each run to check orientation.

The on-shore beach profile is obtained by setting the instrument up on the base line on one profile range. Then, if adjacent profile ranges are 500 feet away or less, three ranges may be surveyed with one set-up.

The plane table sheet is plotted to suitable scale - preferably larger than the scale of the water soundings to obtain more accurate data. (The beach face is considered the most critical area when a beach changes.) The sheet is then oriented, and a rod-man with a stadia board walks along each profile range aligning himself on range by eye from the base line to the limit of backrush, taking significant elevations.

On the left and right profile ranges, elevations are plotted by line of sight, but on the center profile range, elevations are located by stadia. The elevation and stadia notes are usually kept in a separate surveying notebook in the regular manner.

On one survey, both transit and plane table methods were used simultaneously with the same rodman on the beach face. The elevations varied a maximum of 0.2 foot, and the locations varied within a pencil line width when plotted at a scale of one inch equals 250 feet. These variations are acceptable.

For the above three instrument methods, ranges and a control net may - for example, be established as follows:

At Santa Margarita River Beach a testing area for LVTs and DUKWs was established. (Wiegel, 1949) (VI D-12). Using transit and tape, a baseline was established approximately parallel to the shore with profile ranges at right angles to it. Three men made up the survey party - an instrument man and two chain men. Profiles were obtained at about two week intervals for a period of eight months. Three ranges 500 feet apart were established, and also an instrument station 500 feet south of the south profile range. On each range, permanent range poles were set up - consisting of 2 by 12 inch timber painted alternately with black and white one-foot bands. These poles projected above ground from 10 to 20 feet, and were spaced according to the topography from 200 to 400 feet apart on the ranges.

For other U. S. Pacific Coast beaches, it proved satisfactory to place the instrument station at least 500 feet from the nearest profile range, and not over 1,500 feet from that profile range.

For reconnaisance surveying - as by UDTs, instrument-control of the survey is not normally possible. As described previously, horizontal control is usually maintained by judgment and estimation only. On the moderately steep beaches of Monterey, California, UDT one (Bascom 1950b) (VI D-3) measured profiles in this manner, and its results compared favorably with controlled methods described above. The surf zone was about 100 yards wide. On the other hand, some difficulty might arise in making this type of reconnaisance survey on a very flat beach - such as that at Clatsop Spit, Oregon - where the surf zone is about 600 yards wide at times. Surveys on

such flat beaches are important because there are often several bars. In these cases, the UDT reconnaisance survey could profitably be compared with vertical aerial photographs for a more complete knowledge of the beach. The bars could then be located by the photographs, and described by the UDT reconnaisance.

Additional information may be obtained by swimmers. A swimmer should relate his soundings to irregularities, (especially those apt to appear in aerial photographs - such as discontinuities of bars, rip currents, cusps, etc).

4. Office Work:

The office work is done in three steps for the field methods discussed above: reduce field notes, plot the data, and reproduce the finished survey sheets. The recorded soundings are subject to two corrections, one for tide stage and the other for wave height. In the absence of a suitable tide record, the United States Coast and Geodetic Survey's tide tables may be used with interpolations according to their formulas. Possible errors in the predictions (caused by changes in atmospheric pressure, winds, freshets and droughts, and other factors - see section of this manual on tides) usually are neglected. The correction for wave height outside the breakers is presumed to be one-third of the estimated height of the wave following the trough in which the sounding was made (as in Figure VI D-1). If the plane table method has been used, these computations may be made and the soundings corrected directly on the plane table sheet as in Figure VI D-14. Once shoreward of the breaker zone the correction for wave height does not apply, and the correction for the tide stage only is used. The datum used for the survey is usually the local tidal datum. The recorded soundings are called "minus", the correction for wave height "minus" and the correction for tide stage, the same as the tide ("plus" or "minus"). These results are then added algebraically to obtain the corrected sounding. As mentioned previously, the accuracy of surveying in and near the surf zone can hardly be expected to be better than plus or minus one foot. Nevertheless, in order to minimize accumulation of errors from the many corrections to individual soundings, the computations and plotting of the soundings should be to the nearest one-half foot. The beach land profile and elevation notes are reduced by standard surveying methods and this information is plotted on the final profile sheet. For the UDT methods locations and depths are scaled off; for the transit methods, locations of the soundings may be plotted graphically, using a vernier protractor, or computed useing elementary trigonometry. For the plane table method, however, the profile sheet and the plane table sheet should have the same scale so that the soundings can be located on range by tracing. The beach land profile (at a larger scale) may be laid off with proportional dividers. In both cases depths are plotted by making a point at the proper vertical scale.

One of the best ways to reproduce the survey sheets is by ozalid process. When surveys are repeated periodically, transparent ozalids may be used with all the information, such as ranges and lettering already plotted. To finish the survey sheet all that remains to be done is to plot the soundings, assign a drawing number, and place the date of the survey on the sheet. One of these transparent ozalids is shown (Figure VI D-15) and also a finished profile sheet (Figure VI D-16).

5. Summary and Recommendations:

Perhaps the above methods sound unduly complicated. Perhaps a special situation cannot be surveyed in any of the manners described above. Or perhaps better methods have been omitted. At any rate, the foregoing methods all have been used extensively. Individual cases may require special considerations.

Certainly during wartime, elaborate surveys of enemy-held beaches cannot be conducted. Even during peacetime, diplomatic considerations may preclude surveys over certain areas. However, much information can be derived from an aerial sortie,

especially if it can be combined with a UDT sortie. Significant notes may even be recorded by an observer walking along the beach.

If facilities are available, periodic checks of the UDT training surveys with more precise methods of control might prove helpful. For checking control only, one of the transit or plane table methods could be used to fix the position of the swimmer each time he signalled a sounding.

The choice of method for a particular survey will depend on local conditions. For example the two-transit method was used on the Amphibious Oceanography project for surveys on Clatsop Spit, Oregon - where sounding ranges extended about 3,000 feet to sea, and where the strong littoral current prevented the sounding craft from maintaining range (Figure VI D-12). On the other hand, the one-transit method was used at Monterey and Oceanside when the weather was inclement, but otherwise, the plane table method was preferred.

The plane-table method was generally preferred - if the required accuracy could be maintained - for the following reasons:

(i) data is obtained more directly and more easily

(ii) time required for field and office work is shorter

(iii) although the plane table method is basically less accurate than the transit method, to many refinements are not warranted since the DUKW is not always exactly on range and also since the plane table method eliminates the change of error in reading the transit vernier and the protractor vernier.

When facilities and local conditions permit, no opportunity should be missed to survey beaches. The actual amount of time and effort required are relatively small. For example, at Oceanside, California - on three established ranges - the required time in the field was only:

By the plane table method -
about 45 minutes for water soundings and about 35 minutes for beach profiles

By the one-transit method -
about 80 to 90 minutes for water soundings, 50 to 60 minutes for beach land profiles.

The time in the office was also shorter for the plane table method - about 2 hours for complete reduction of notes and checking and completing profile sheet, compared to $2\frac{1}{2}$ hours by the transit method.

The total direct man hours required for surveying and plotting profiles on the three ranges at Oceanside was about 7 hours for plane table method, $10\frac{1}{2}$ hours for one-transit method.

These time records were achieved by relatively inexperienced personnel with only about six training days under a civil engineer officer. The instrument man had had only previous rodman experience; other personnel no previous surveying experience. The accuracy of the surveys has been illustrated in Figure VI D-7.

6. References Cited: RESTRICTED Security Information

VI D-1: Bagnold, Major R. A. (1940) "Beach Formation by Waves; Some Model Experiments in a Tank", Paper No. 5237, Journal of the Institution of Civil Engineers, London, No. 1, 1940, pp. 27-52.

VI D-2: Bascom, Willard N., (1950a) Tests of Landing Craft at Monterey Bay, California, University of California, Berkeley, Report #155-20, Contract N7onr-29519, April 1950. (RESTRICTED)

VI D-3: Bascom, Willard N., (1950b) Underwater Demolition Team at Monterey, University of California, Berkeley, Report #155-24, Contract N7onr-29519, May 10, 1950, (RESTRICTED)

VI D-4: Bowditch, Nathaniel, (1943) American Practical Navigator, (Report H. O. -9), U. S. Hydrographic Office - under the authority of the Secretary of the Navy, Washington, 1943, (Chapter XVII).

VI D-5: Bulletin of the Beach Erosion Board (1947): Vol. 1, No. 2, July 1, 1947, pp. 1-7.

VI D-6: Caldwell, J. M., (1950) "Supersonic Sounding Instruments and Methods", Proceedings, American Society of Civil Engineers, Vol. 76, Separate Number 50, Dec. 1950, 15 pp.

VI D-7: Fielding, T. P. (1950), Report of UDT Operations in Monterey Bay Area, to CO, UDT One, March 31, 1950. (RESTRICTED)

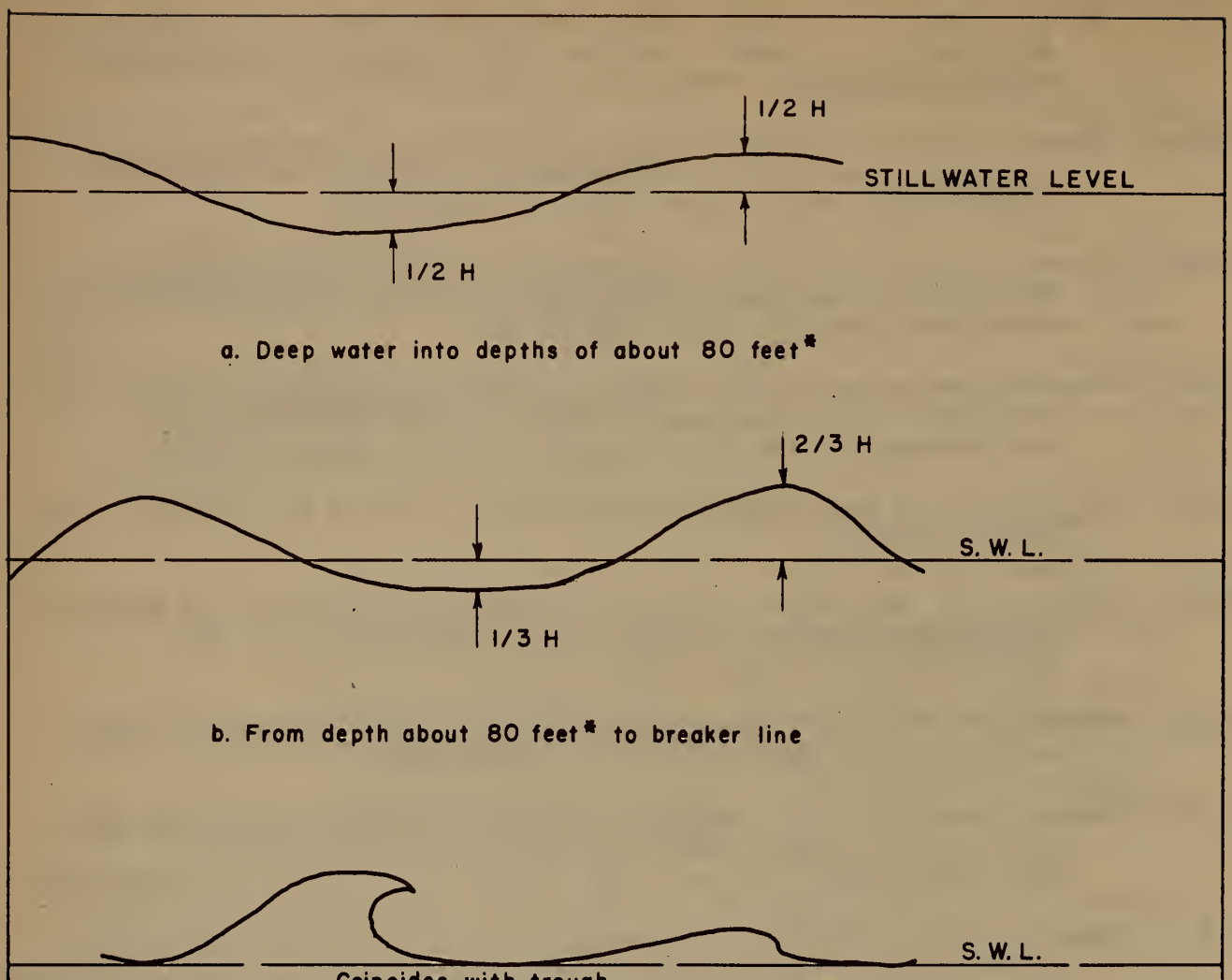
VI D-8: Isaacs, John D., (1945) Report on Beach Survey with Sea Sled and Mast at Pismo, California, University of California, Berkeley, Report HE-116-135, Contract NObs 16290, July 12, 1945. (RESTRICTED)

VI D-9: Isaacs, John D., & Willard N. Bascom, (1949) "Water-Table Elevations in Some Pacific Coast Beaches" Transactions, American Geophysical Union, Vol. 30, No. 2, April 1949, pp. 293-294.

VI D-10: Munk, Walter H. (1949) "Surf Beats", Transactions, American Geophysical Union, Vol. 30, No. 6, Dec. 1949, pp. 849-854.

VI D-11: Patrick, Donald A. (1950), A Comparison of Beach Elevations at Limits of Backrush and Uprush with USC&GS Tide Predictions on Several Pacific Ocean Beaches, University of California, Berkeley, Report 155-36, Contract N7onr-20519, Oct. 5, 1950. (RESTRICTED)

VI D-12: Wiegel, Robert L., (1949) Landing Craft Test Operations at Santa Margarita River Beach, Oceanside, California, University of California, Berkeley, Report 155-1, Contract N7onr-29519, April 18, 1949. [redacted]




a. Deep water into depths of about 80 feet*

b. From depth about 80 feet* to breaker line

c. In surf zone

## APPROXIMATE RELATIONSHIPS BETWEEN WAVES AND WATER LEVEL USED FOR HYDROGRAPHIC SURVEYING

*About 80 feet for waves, T=12 seconds. The transition point for correction of 1/2 H to 1/3 H has been taken at $d/L_o = 0.11$, so actual depth for changing corrections for waves of T≠12 seconds is $T^2/12^2 \times 80$.



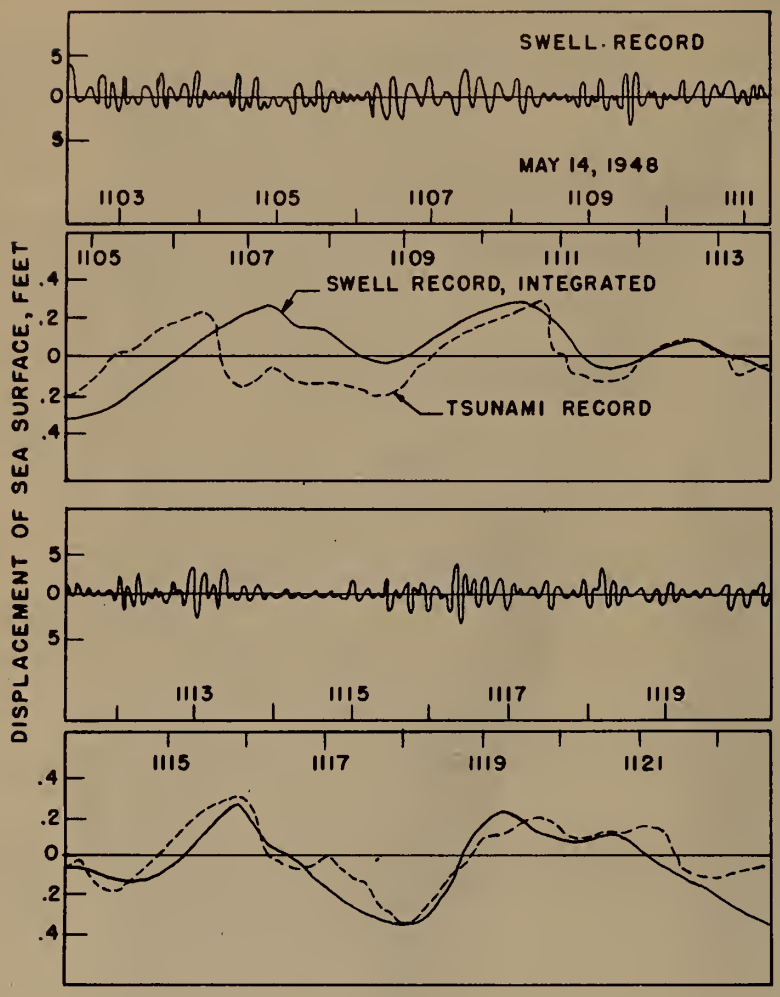
FIGURE XII D-1




FATHOMETER SURVEY RECORD
CARMEL 0+00
JULY 1, 1947

BLACK LINE PIPS AT EVEN 200 FOOT INTERVALS PUT ON AT RADIO COMMAND OF TRANSIT ON BEACH WHICH TURNS PRECALCULATED ANGLES

FIGURE VII D-2




Detailed comparison of waves on swell and tsunami records; the upper time scale refers to swell record and integrated swell record; the lower time scale refers to tsunami record and is displaced 140 seconds relative to upper time scale to correct for relative locations of recorders offshore. (Munk, 1949)

FIGURE VI D-3



SEARCHING FOR THE LIMIT OF BACKRUSH (ZERO SOUNDING)
AT THE COMPLETION OF A RUN OF SOUNDINGS

FIGURE VI D-4

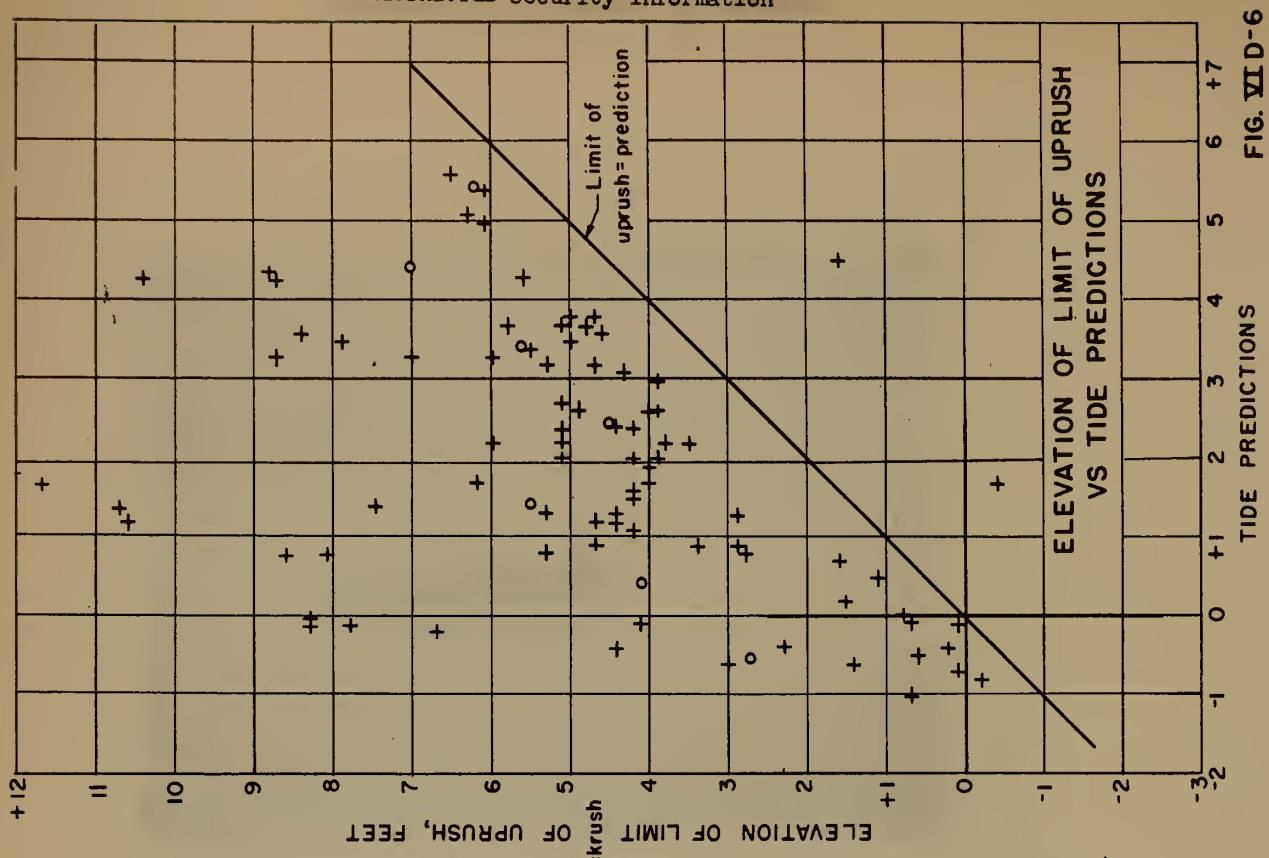



FIG. VII D-6

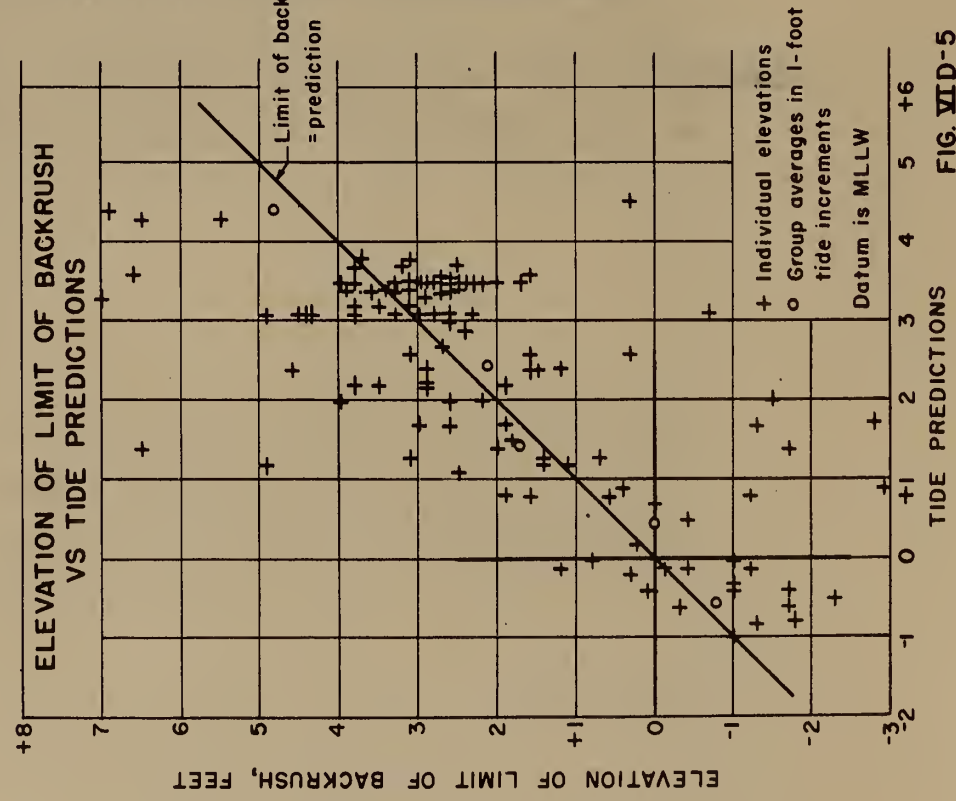



FIG. VII D-5





FATHOMETER INSTALLED IN DUKW

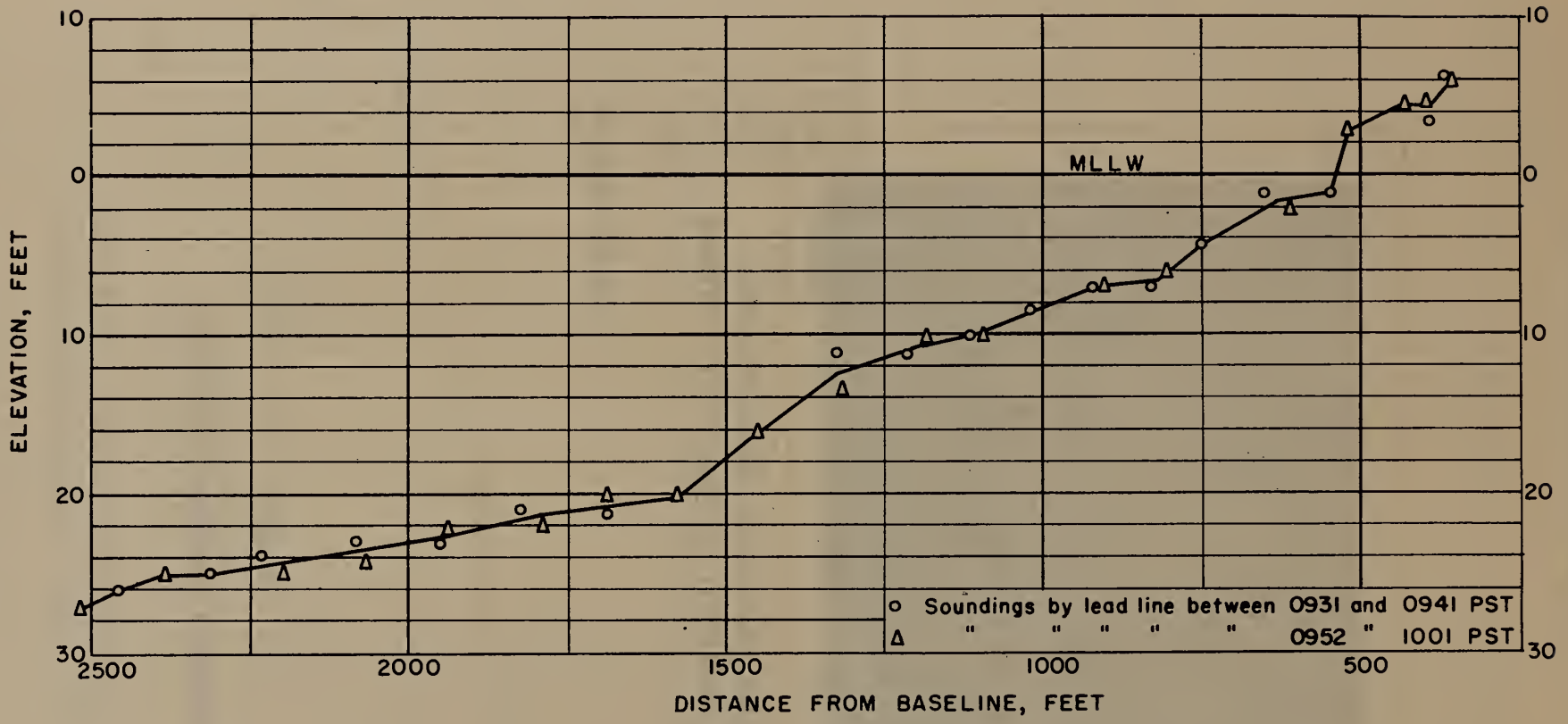
FIGURE VII D-7





SOUNDINGS BY HAND LEAD LINE FROM A SMALL BOAT

FIGURE VII-8




Control by plane table method
From survey at Santa Margarita River, Oceanside, Calif.,
on range 12+50, 17 Jan 50

TIDE PREDICTIONS:

| PST | Height |
|---|---|
| 0145 | 2.2' |
| 0754 | 7.2' |
| 1514 | -1.6' |
| 2144 | 4.4' |

COMPARISON OF ACCURACY FOR LEAD LINE SOUNDINGS

FIGURE VII D-9

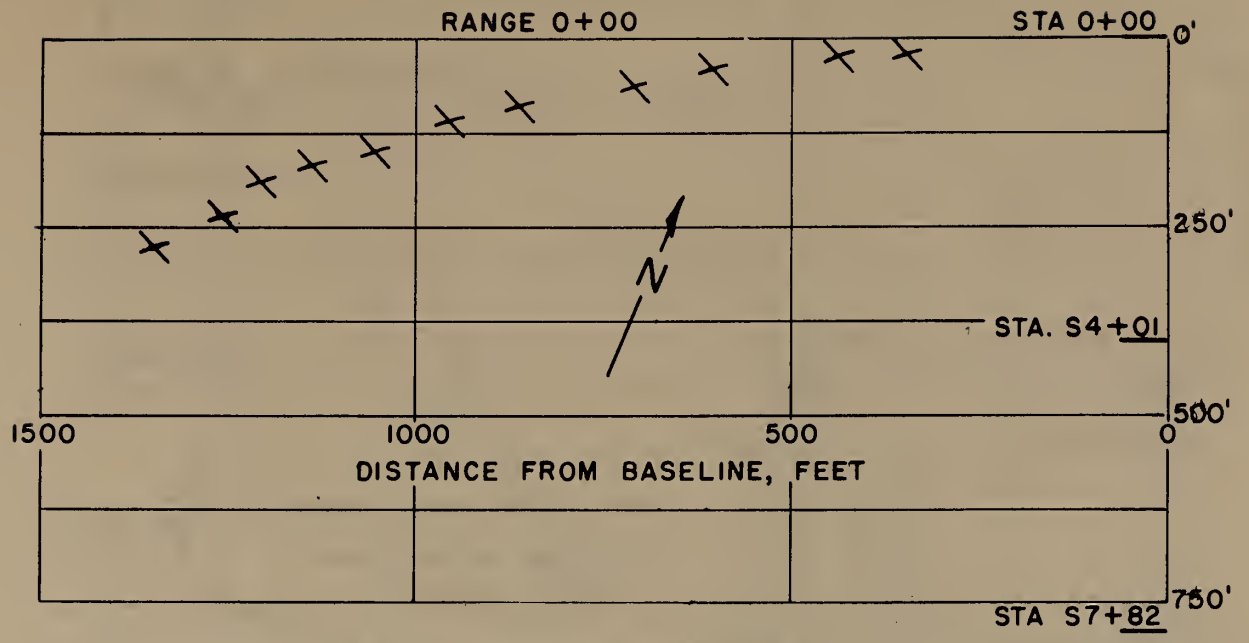


RESTRICTED Security Information



FOUR SEA SLEDS IN BACKGROUND
WITH MASTS PAINTED IN ONE FOOT STRIPES

FIGURE VII D-10



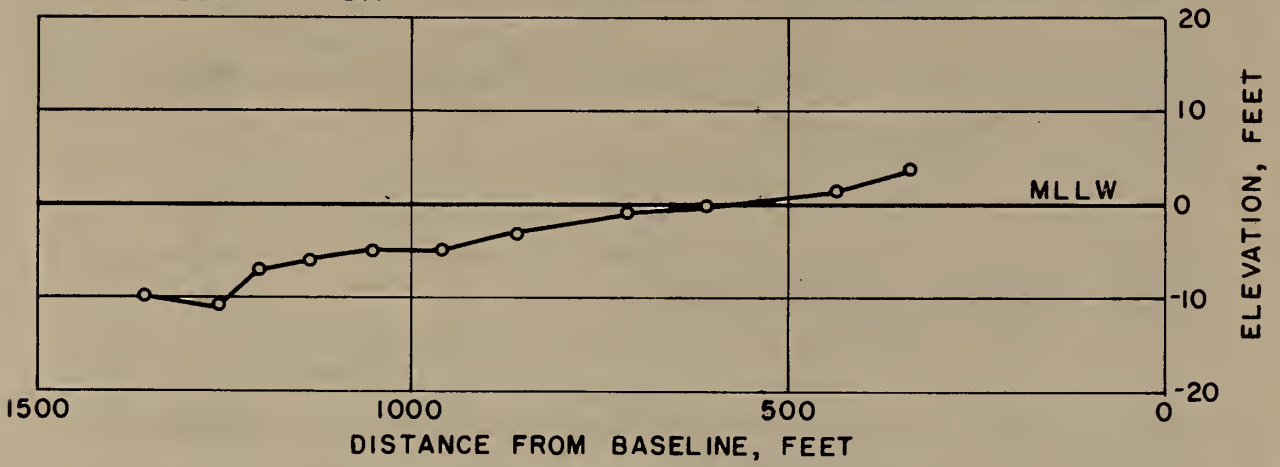

PLAN POSITIONS OF SOUNDINGS

TIDE STAGES:

| PDT | Height |
|---|---|
| 0119 | 8.8' |
| 0817 | -1.6' |
| 1437 | 6.6' |
| 2004 | 2.1' |

Control by two-transit method

o Elevations by sea sleds using vertical angles



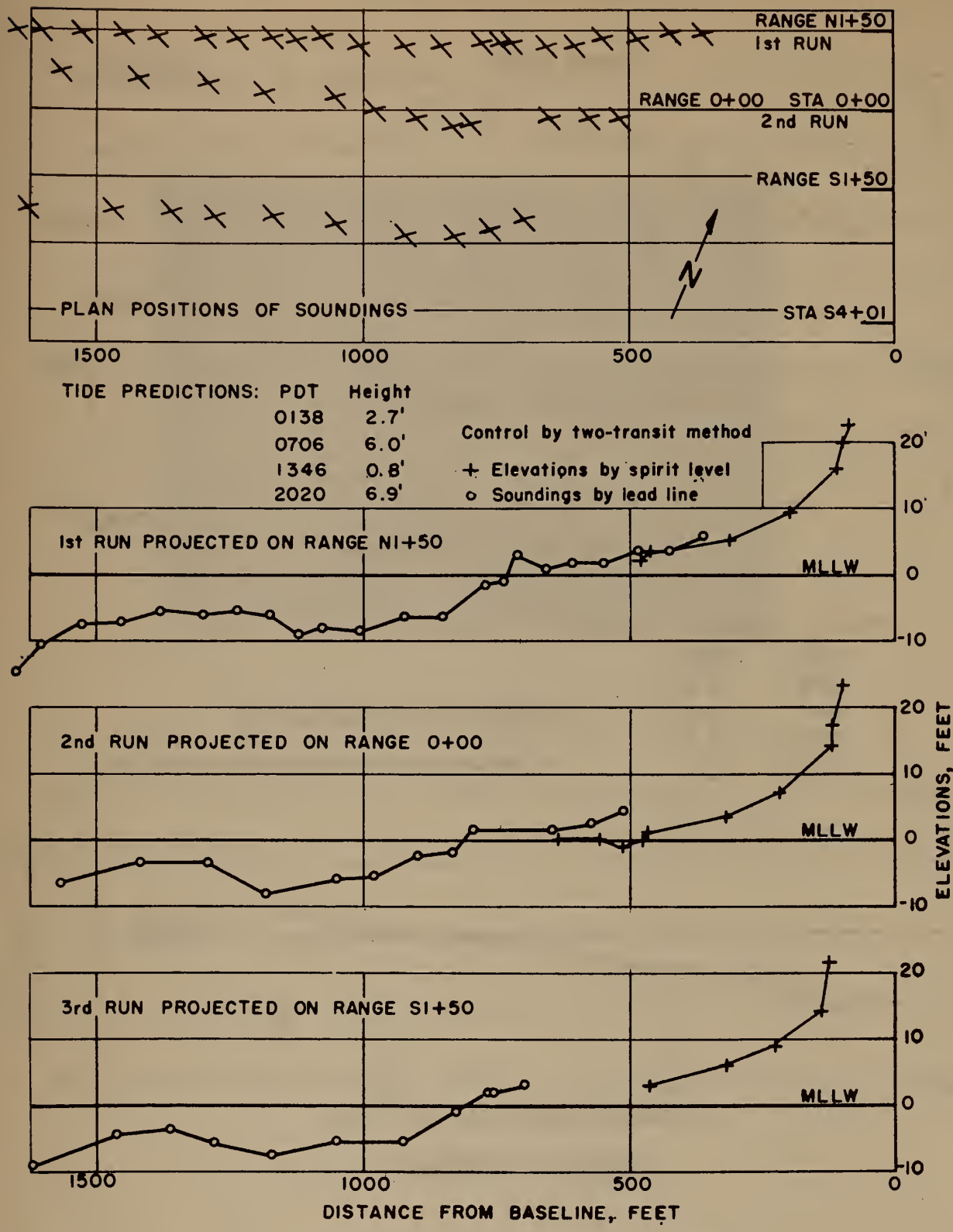

PROJECTED PROFILE

## BEACH SOUNDINGS AND PROFILE

CLATSOP SPIT, OREGON; 3 MAY 50



FIGURE VII D-11

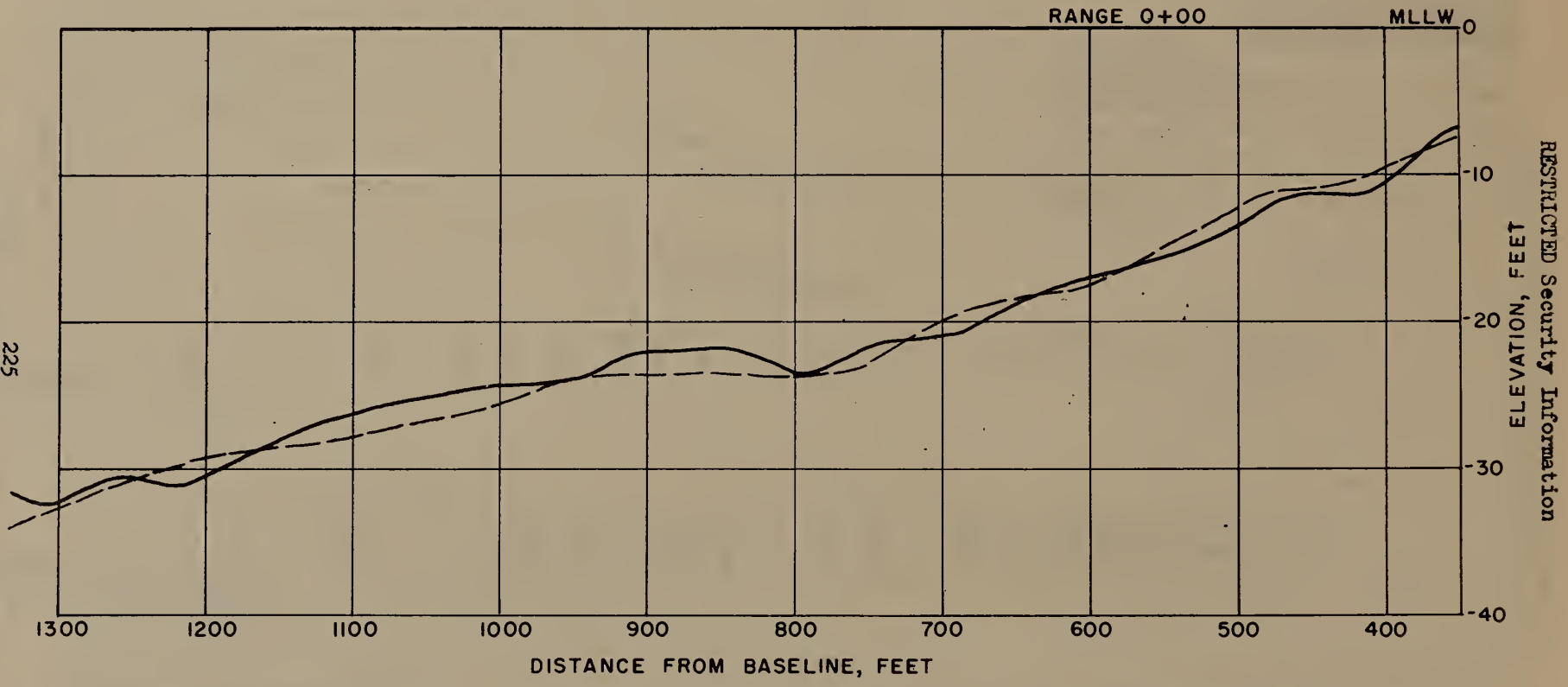



## BEACH SOUNDINGS AND PROFILES

CLATSOP SPIT, OREGON; 25 MAY 50

RESTRICTED Security Information





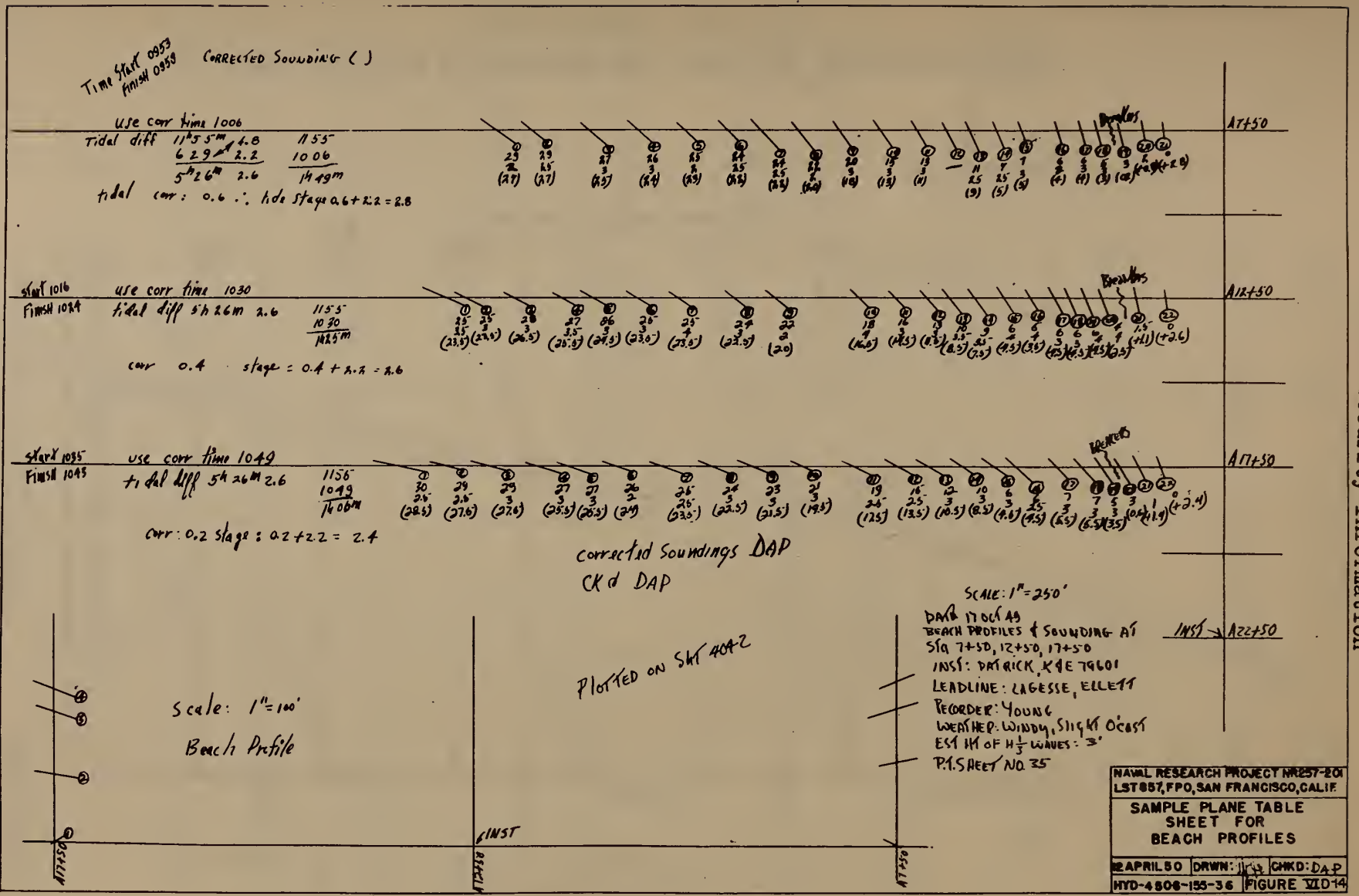

RESTRICTED Security Information

COMPARISON OF FATHOMETER AND LEAD LINE SOUNDING METHODS

Carmel, California; 1 July 1947

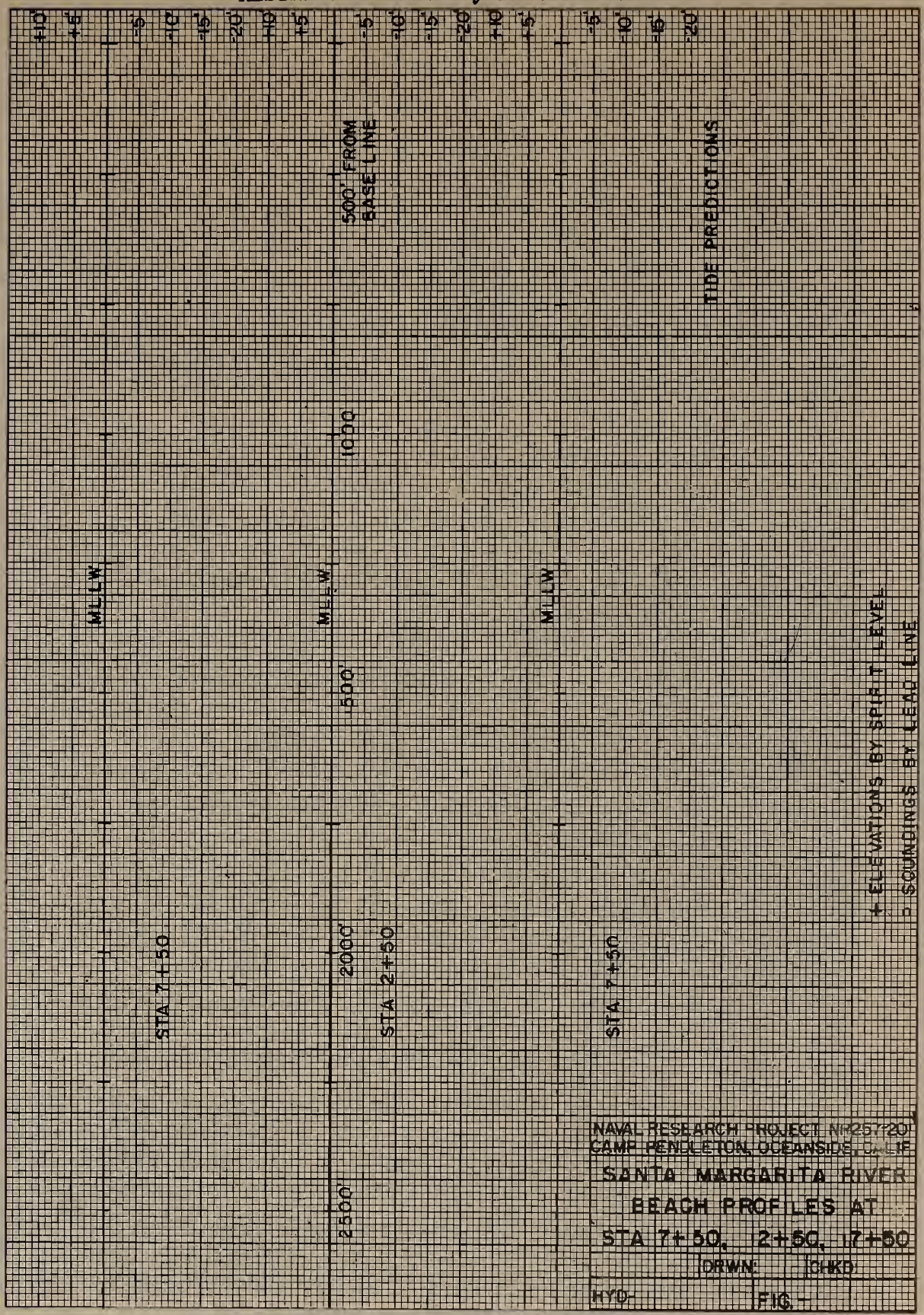
FIGURE VI D-13

RESTRICTED Security Information




| NAVAL RESEARCH PROJECT NR257-201 LST 857, FPO, SAN FRANCISCO, CALIF. | | |
|---|---|---|
| SAMPLE PLANE TABLE SHEET FOR BEACH PROFILES | | |
| 12 APRIL 50 | DRWN: | CHKD: DAP |
| HYD-4506-155-36 | FIGURE VII D-14 | |

RESTRICTED Security Information

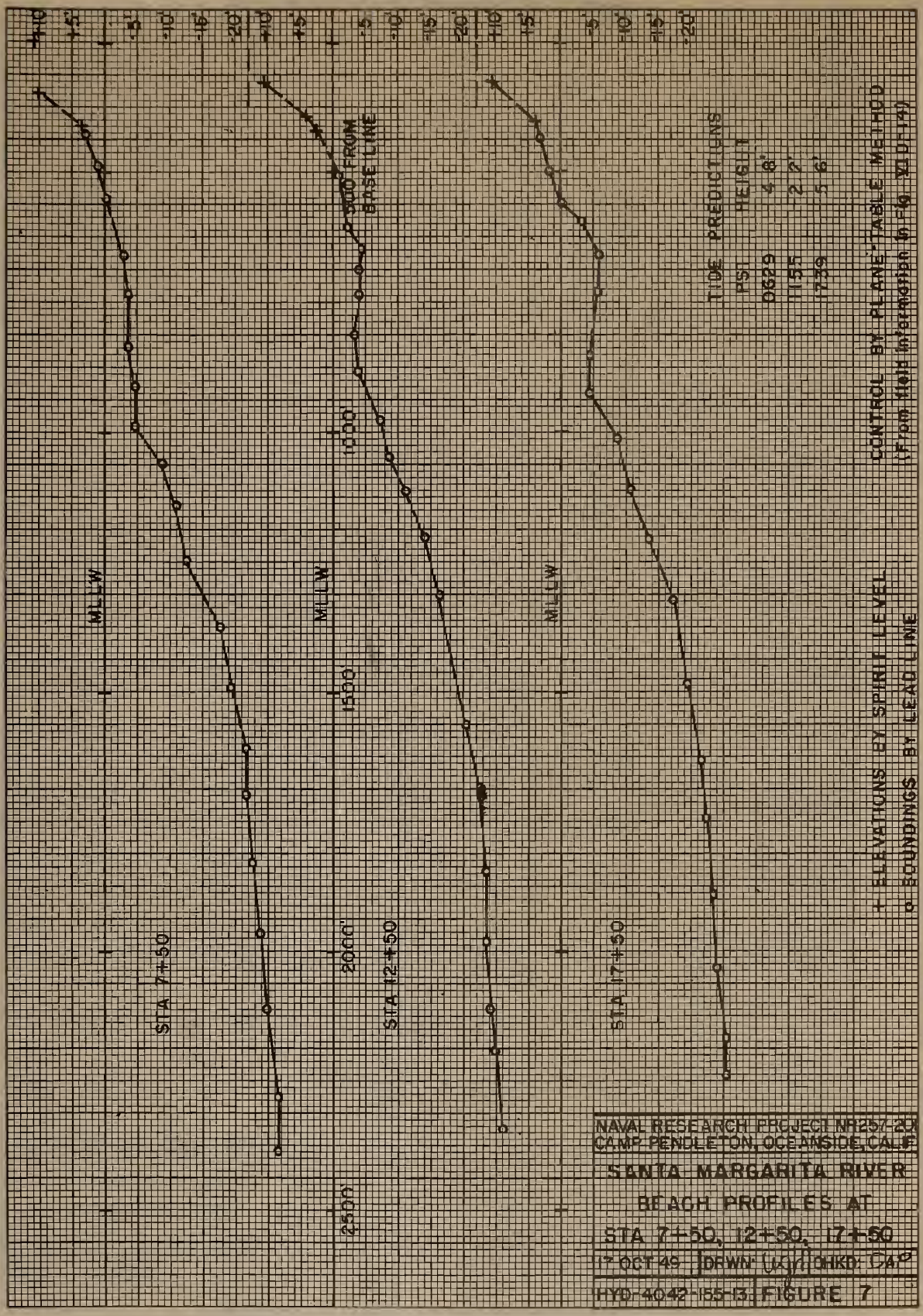



SAMPLE TRANSPARENT OZALID





FIGURE VII D-15




SAMPLE COMPLETED PROFILE SHEET

# E. WAVE MEASURING INSTRUMENTS

BY
F. E. SNODGRASS

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1. Early Techniques and Instruments to Measure Ocean Waves:

a. Visual Methods Used Until 1945:

The data and methods of wave measurement used before 1904 are summarized in the book Wave Action by D. D. Gaillard (Reference VI E-16). These early data are incomplete, inaccurate, and of little use to the engineer.

Other than visual observation, Gaillard only mentions measuring waves by aneroid barometers. In this method, delicate aneroid barometers were attached to a fixed position on a ship, and these measured the variations in atmospheric pressure as the ship rode over the swells. The probable error in measuring wave heights by this method was assumed to be between 2 and 3 feet. The wave periods were determined by counting the number of waves passing a fixed point on the ship in a given time.

Several techniques were developed to improve visual observations of ocean waves. Three of these methods were: the Plumb-bob method; the float and transit method; and the beach pole method.

(1). Plumb-bob Method: (Reference VI E-49)

> "Breaker heights can be determined easily by means of a Plumb-bob. A suitable weight is suspended by a strong string, and the trough and crest heights of the breaker are determined with respect to the known elevations of a railing or flooring on a pier. In making measurements, the observer is stationed at about the outer line of breakers. After a few trials, the observer can set the bob at the trough level and raise it in order to just touch the crest of the following waves at point of breaking. The length of string taken in is the wave height. Periods are determined by observation from the pier of waves as they pass a selected location."

(2) Float and Transit Method: In this early technique, readings (of the motion of the mast attached to an anchored float) were obtained by sighting on the mast through a transit placed on the end of a pier. The upper and lower position of the float during a given cycle was determined by the movement of the mast flags past the horizontal hairline in the transit. The wave period was determined by timing 10 upper positions of the float as observed through the transit. The recorded readings consisted of the estimated upper and lower float positions for a series of consecutive waves during a noted period. A separate set of readings gave the wave periods. Analysis of the curves indicated that the observed wave crests, troughs, and periods corresponded to those for the waves generated by local winds. Although the effect of the underlying swell was observable, determination of its magnitude and period were very difficult.

(3) The Beach Pole Method: The beach pole method, described in a University of California report (Reference VI E-25) was a method of estimating the height of waves in the surf zone. A light-weight pole was used, from 6 to 15 feet long, plainly graduated in feet and tenths of feet. One observer held the pole vertically at the limit of backrush, while a second observer raised or lowered the position of his eye until the waves appeared to touch the horizon at the point of breaking. (See Figures VI E-1). The observer read the marking on the rod at this instant. Since the line of sight was horizontal, and the limit of backrush was assumed to be the still water level, the beach-pole reading

was the height of the breaker crest above still water level. It was necessary to apply a correction factor to this observed height to obtain the height of the breaker above the preceding trough.

A plot of the observations (taken by K. C. Reynolds in the Beach Erosion Board tank) showed that the height of the breaker above the preceding trough was approximately 1.5 times greater than the breaker height above still water level.

Beach poles have been used for wave height measurement, but their use is limited to relatively flat beaches where the position of the backrush is more nearly constant. Observations have been made regularly at Coronado with modified sighting or beach pole. In general, however, use of the beach pole has been discontinued as it proved unsatisfactory for steep beaches. Full description of the equipment and specifications is given in the report Instructions for Use of Beach Poles in Measuring Wave Heights (Reference VI E-25).

b. Early Wave Measuring Instruments:

After the beach pole method (of measuring breaker height in the surf) was discarded, other methods were developed which included:

(i) The Sighting Bar

(ii) The Fathometer, which led to the development of Echo Sounders

(iii) Surface Type Recorders

(iv) Pressure Type Recorders

(1) Sighting Bar Breaker Meter: (Reference VI E-23)

Developed in 1944, its operation depended upon the geometry of sighting the wave against a vertical calibrated scale, as shown in Figures VI E-2 to 4. A sighting bar meter consists of a rigid base which holds two vertical supports, one containing the eye piece. The sighting edge, set at 24 inches, serves as a reference base line for calculating results.

Sighting bar investigations revealed certain discrepancies between a calculated breaker height and the actual breaker height. These discrepancies are fully discussed in several University of California technical reports, a summary of which was written by Isaacs (Reference VI E-20).

(2) Echo Sounders: A fathometer which recorded the variation of water depth as the boat floated on the surface was tried as a wave-measuring device. Although this method proved unsuccessful because of tilting of the boat and operational difficulties, it led to the development of an inverted echo sounder.

In the inverted echo sounder, the sonic transmitter was placed on the bottom (or in some other fixed submerged place), and it then transmitted a sound impulse to the surface. The sound was reflected at the surface and the echo was picked up by the sonic receiver. The signal was then converted by the receiver into an electrical impulse, which was then amplified to produce a suitable record. The time of travel of the sound impulse was converted to a measurement of wave height.

In 1941, the Beach Erosion Board made preliminary tests of inverted echo sounders, using a Hughes echo sounder. These tests were made only in tanks with still water; (Reference VI E-2). Earlier tests had been made in England, (Reference VI E-6). This method was found to be generally unsatisfactory due to the relatively large angle of the sound cone and the general inconvenience of the electronic equipment.

(3) Surface Type Recorders:

(a) Float-operated Type: The first experimental instruments developed for recording ocean waves were simple mechanical systems. One of these utilized a surface float that was restrained in its horizontal movement, but not in its vertical rise and fall. This float was linked to a recorder mechanism which furnished a permanent record. The device was simple, direct writing, and required no power.

The disadvantages of the float recorder were:

-that it required a supporting structure;

-as the mechanism interconnecting the float and recorder was subjected to frequent and rapid reversals of direction, it was difficult to make a suitable design; and that

-the frequency-amplitude response was poor; the response was generally unsatisfactory for short period waves if the equipment was constructed to operate successfully for high waves.

The Scripps Institution of Oceanography used a float-operated recorder on its pier at La Jolla, and the Coast and Geodetic Survey (in cooperation with the New Jersey State Board of Commerce and Navigation) operated a similar instrument at the Million Dollar Pier at Atlantic City. (Reference VI E-45).

(b) Parallel Wire Type: (Reference VI E-2)
Forerunner of the step-resistance gage used today, this type has two conducting wires (electrodes) mounted vertically, with their bases below the lowest trough and their tops above the highest crest. A voltage is impressed across the two wires, and the resulting current is measured on the recorder. As the wave rises, the current flow increases due to the increased conducting path furnished by the water. By proper arrangement of the wire size, wire spacing, impressed voltage and frequency - the current can be adjusted to vary directly with the wave height over a considerable range.

These gages had a simple circuit: essentially an alternating voltage source, a recorder, and a pair of wires. No amplification of the signal was required. The gage was unsatisfactory, however, because electrolysis destroyed the electrode wire under sustained operation; the straight-line calibration held over a very limited time; and the structural durability of the instrument was low.

The parallel wire gage is used today only as a laboratory instrument for recording waves in model studies. (Reference VI E-53)

(4) Pressure Type Recorders: Theoretical mathematical studies established a relationship between the surface wave and the sub-surface pressure fluctuation, and this relationship is the basis of many wave recording instruments. The first pressure recorders were mechanical-hydraulic instruments. Later types converted the pressure fluctuations to electrical signals that could be remotely

recorded. A Russian installation used an air-pressure chamber connected by a length of 2 cm. pneumatic hose to a pneumatic-type shore-based recorder. (References VI E-2 and 4) Length of connecting hose could not exceed 80 to 100 meters, and the maximum usable depth was 6 to 8 meters for a wave 2 meters high.

(a) Brass Box: In 1945, the Woods Hole Oceanographic Institution developed a pressure-time recorder consisting of an Esterline-Angus chart drive with a special pressure-actuated pen. (Reference VI E-24). The recorder, enclosed in a brass container, was lowered into the water. It had to be retrieved periodically to rewind the clock drive and change the chart. The instrument was designed so that wave frequency differential pressures actuated the pressure element. It did not record tidal cycles or changes in mean depth. When it was raised or lowered in the water, rapid large pressure fluctuations caused a pressure differential (between the tube and element and the inside of the case) which exceeded the allowable differential pressure of the element. These pressures were equalized by a relief valve set to open at differential pressures between the case and tube slightly greater than the range of the elements used. A "slow leak" (3-inch length of small bore tubing) relieved long-period, tidal pressure changes. The slow leak was essentially to maintain the case pressure to correspond to the mean depth of water over the instrument. The Brass Box Recorder is shown in Figures VI E-5 and 6.

(b) Cambridge Recorder: This recorder, obtained under lend-lease from England, was subjected to laboratory calibrations and tests by the University of California, (Reference VI E-17). The original equipment consisted of an underwater pressure pick-up unit, but no recording mechanism. The British used a galvanometer of 2-second period and a recording camera to record the electrical output of the pressure pick-up unit.

In the laboratory calibration, a Weston Model No. 721 photo-electric potentiometer and an Esterline-Angus 5 milliampere recording milliammeter, was used to record the pressure pick-up output. With this equipment, the range of maximum differential pressure was about 6 feet of water. The underwater potentiometer had 6 leads coming from it, which were connected to a Shakespeare micrometer and the solenoid-operated valve. The Shakespeare micrometer (which detected the aneroid movement due to pressure changes) was a small hot-wire bridge, whose balance was disturbed when the tension changed in the resistance arms. The bridge was supplied with 250 ma. DC, adjusted accurately on the control box meter. The other two leads from the bridge, after being fed through a resistance network (which standardized the sensitivity) were attached to a mirror galvanometer or other sensitive indicating device.

(c) Rheostat Recorder: In 1945, the University of California constructed a differential pressure recorder (which consisted of a rheostat type pressure head and a recording milliammeter) and subjected it to laboratory and field tests. (Figures VI E-7 and 8.) The equipment consisted of an aneroid-type pressure element mounted in a fluid tight chamber. The average surrounding pressure was applied to the inside of the aneroid element; and the fluctuating pressures due to wave action were applied to the chamber. The movement of the aneroid element (which depended on the differential pressure), was transmitted from the pressure chamber through a sealing sylphon bellows to the moving arm of a potentiometer. Sea water was separated from the instrument parts by using loose rubber diaphragms. The average pressure was obtained by inserting a series of felt disks in the pressure line which was connected to the inside of the aneroid element. The position of the rheostat arm controlled the deflection of a recording pen on the strip chart

of a recording milliammeter. DC power was supplied by a small battery or rectifier. The unit was designed to submerge to about 150 feet maximum depth, and to operate with about 5 mile lengths of connecting wire.

The Rheostat Recorder (later called the Mark I recorder), proved unsatisfactory because of mechanical difficulties.

(b) Strain Gage Recorder: (Reference VI E-18).

The bronze case of the Rheostat recorder was then used for a strain gage transducer called the Mark II Shore Wave Recorder. This unit consisted of a strain gage type underwater pressure pick-up unit and a standard Foxboro SR-4 time strain recorder. (Figure VI E-9). The pick-up unit was connected to the recorder through a 3 conductor cable.

The pressure pick-up unit had a bronze case with the working units mounted inside. The pressure of the sea water was transmitted to the inside of the sylphon bellows through a loose rubber diaphragm which kept the sea water away from the bellows. The space between the diaphragm and the bellows was filled with fresh water. The sea water pressure - transmitted through the bellows - deflected the free end of a cantilever beam, and the resulting deflection was proportional to the pressure. While the unit was sensitive to total pressure, the recorder showed the difference between the instantaneous total pressure and the arbitrary reference (established by the "zero" adjustment control on the recorder).

The beam deflection was measured by two strain gages. Standard SR-4 Resistance strain gages were used - cemented on each side of the cantilever beam - and these two strain gages formed two of the arms of a bridge circuit. The other two arms were incorporated in the recorder, forming part of the balancing mechanism for controlling the position of the pen arm. The use of strain gages for two of the bridge arms compensated for errors otherwise resulting from temperature changes. They also doubled the sensitivity of the instrument.

The beam was made from high-strength, aluminum-alloy material. Its geometrical proportions allowed a maximum pressure of 25 feet of sea water. Pressures up to this level produced deflections below the elastic limit of the material of the beam. To protect the beam from overstress, in case of excess pressure, limit stops for the bellows motion were built into the equipment.

In laboratory and field tests of the first model, the Mark II Shore Wave Recorder proved satisfactory for its intended use within its design limit (maximum of 25 feet of sea water). Dynamic characteristics were predicted from the static calibration.

(c) The Sea Sled: (Reference VI E-19)

This is included here (although it is not a wave-measuring instrument) because it is used to carry recording devices, rods for stadia sounding, and lines through the surf. The sea sled (Figure VI E-10) is propelled along the bottom through the surf zone by currents resulting from wave action. For a seaward trip, it is rolled down the beach to the backrush and set free. The uprush of the water makes its wheels sink into the sand, but as soon as the skids rest on the sand, this sinking ceases. During the uprush, the fluke digs into the sand, thus preventing the sled from being washed shoreward. The subsequent backrush of the water lifts the gate and the sled proceeds seaward at from 1-8 knots until it encounters another wave. Then, the gage drops back to its lower position, the fluke digs in, and the sled remains on the bottom until another backrush begins.

Use of the sea sled is restricted to fairly smooth beaches of reasonably firm sand or small gravel. While it will travel through moderate surfs, it functions best in a heavy surf. Presumably a sea sled could be constructed for surf of any intensity.

The following describes a typical sea sled operation: (Reference VI E-19)

"In twelve minutes it had travelled to 700 feet, in fifteen minutes to 900 feet, and in seventeen minutes to 1200 feet. It then stopped and after an additional 20 minutes had not moved further. It was decided to retrieve it but immediately upon attempting to drag it in, it resumed travelling and travelled continuously at a rate of 2 to 6 knots to a total distance of 1600 feet, which was all the line available. It was impossible to stop it and the single operator was dragged into the surf before help was obtained. An automobile was used for retrieving the sled. The total travel from the backrush was 1700 feet. The sled had apparently encountered an obstacle at 1200 feet or had become turned partly around, and was straightened out when pulled shoreward. Despite the littoral drift the line was within 100 feet of a perpendicular to the beach."

2. Present Day Instruments:

a. Performance Tests of Eight Wave Recorders in 1947:

The Beach Erosion Board in May 1947, tested the following wave gages in order to compare their performance - in operation, installation, and servicing. The first three are underwater wave measuring instruments, the rest are surface type.

(i) Mark III Shore-Wave Recorder (University of California).

(ii) Woods Hole Shore-Recording Wavemeter

(iii) Inverted Echo Sounder

(iv) Float-operated Recorder

(v) Parallel-Wire Gage (large wire type)

(vi) Parallel-Wire Gage (small wire type)

(vii) Step-Resistance Gage (series type)

(viii) Moving Picture Camera.

The results and conclusions of these tests are reported in the Beach Erosion Boards An Ocean Wave Measuring Instrument, (Reference VI E-2) as follows:

...None of the gages in the form tested, were satisfactory measuring instruments adapted to a long-term measurement program of surface waves.

...Both underwater pressure gages were satisfactory for the production of records of bottom pressure changes, but the transfer of these records to water surface fluctuations was unsatisfactory.

...The echo-sounder was found to give an erroneous record.

...The parallel-wire gages were not found to be adapted to the study because of structural defects and the fluid nature of the calibration curves.

...The step-resistance gage met all requirements for a short period, but was found to become sluggish and unreliable in continued use.

...Stereo or movie camera records, although giving accurate pictures of the sea surface, were not considered adapted to a long-term measurement program because of processing difficulties.

...It was considered that the step resistance gage held the most promise for development into a satisfactory gage with a minimum expenditure of time and effort.

b. Improved Step Resistance Gages Resulting from 1947 Experiments:

The existing step-resistance gages were therefore modified and improved by the Beach Erosion Board, and two types (series and parallel) were developed which are still used. Further development of underwater pressure recorders, and improvement of procedures for analyzing pressure records have been continued by the Scripps Institution of Oceanography, The Woods Hole Oceanographic Institution, and the University of California Institute of Engineering Research at Berkeley.

The step resistance gage comprises a series of electrical contact points (modified spark plugs) installed along a sealed pipe at 0.2 feet intervals. The spark plugs are connected to a resistance circuit within the pipe. The gage is attached vertically to a supporting structure (such as a pier) with the bottom of the gage below the lowest expected wave trough. The top of the gage must be above the highest wave crest. The normal length of a step resistance gage is 25 feet. (See Figure VI E-11).

A constant voltage transformer (115 volt AC) supplies power to the gage; its primary is converted through a timing switch to provide automatic programming. This alternating current is supplied to prevent polarization, and it is converted (through a selenium bridge rectifier) to a proportional DC current, which in turn drives the recording unit mechanism. A Brush magnetic pen recorder is commonly used, which has a high frequency response and can record the shortest period wave.

The values of the resistors, connected to the gage's contact points, are adjusted so that there is a straight-line variation between the current and the submerged length of the gage. By recording the variation of the gage current, a record of the rise and fall of the sea surface is obtained which includes tide variations as well as wave action. Wind chop and wave form are reproduced accurately, providing the response of the recorder is sufficiently rapid.

At present, three step-resistance gages are in operation along the Pacific Coast. The first was installed at Huntington Beach in May, 1948; the second at El Segundo in July, 1948; and the third at Mission Bay in December, 1949. These recorders have been continuously operated. Periodic cleaning every four to six months has been found necessary for proper operation. Other step-resistance gages have been installed along the Gulf of Mexico and the Atlantic Coast.

(1) Series Type Step-Resistance Gage: In this type, the resistors are connected in a series circuit, with the junctions between resistors tied tothe contact points. As the sea rises, the water shorts all resistors tied to submerged contact points, which causes an increase in current proportional to the number of contacts below the surface.

This instrument is susceptible to current leakage because of the relatively high value of the resistors connected between the contact points. The film of water left on the instrument by the receding water tends to short the resistors connected between the contact points. Because of this, the use of the series type resistance gage is restricted to measurement in fresh water, where current leakage caused by the water film is negligible.

(2) Parallel Type Step-Resistance Gage: In this gage, one end of each resistor is connected to a spark plug, the other end to the gage voltage supply. The sea provides a current path between the contact points and a ground rod. The ground rod is then connected to the other side of the voltage source. As the spark plugs are submerged, the resistors are added in parallel. The values of these resistors are so selected that the current flowing in the gage is proportional to the number of contact points submerged.

The Parallel type Step-Resistance Gage is not affected as seriously by the accumulation of water film as is the Series Type, because its resistance values are small compared to the water film resistance. The gage should be used only in salt water because the resistance path between the contact points and the ground rod must be small compared to the resistor value.

c. Pressure Type Gages:

Surface waves are often measured by recording the sub-surface pressure fluctuations, and translating these fluctuations to the surface by means of theoretical relationships. Discussed elsewhere are the relationships between surface wave characteristics and recorded pressure variations, and the methods of analyzing these pressure records.

To obtain a pressure record, a transducer converts the water pressure fluctuations into electrical current or voltage signals, which can be recorded by suitable electrical or electronic instruments. Various types of transducers designed for this purpose include inductance bridges, strain gages, thermocouples, potentiometers, coils in magnetic fields, and capacitive bridges. The transducer installed in a waterproof case is called a "pressure head". The pressure head is placed somewhere below water surface, and is connected by an electrical cable - often several miles long - to a suitable recording unit located on shore. The pressure type "shore wave recorders" are designed to register pressure variations on a continuous strip chart without distorting the pressure wave form. These systems therefore are designed to respond linearly with pressure and to have sufficiently rapid response to follow the highest frequency components in the pressure variations. The chart paper is divided into uniform rectilinear, or curvilinear, divisions, and the recording mechanism is usually the direct writing type (ink or hot wire). Operation of the system is kept simple because unskilled personnel often attend the shore installation. Automatic programming devices operate the recorders in accordance with prearranged schedules.

(1) Mark III Shore Wave Recorder, University of California (Reference VI E-7) The transducer of the Mark III recorder has a potentiometer linked to a bellows system which is acted upon in one direction by the average pressure through a slow leak, and in the other direction by dynamic pressure.

This transducer responds to wave pressure variations, but not to tide changes. The shore installation comprises a special bridge and electronic power supply and an Esterline-Angus pen recording milliammeter.

The advantages of this recorder are: first that its calibration is linear and independent of wave period and water depth, and second that the system can be operated from 36 volt batteries with a total load current of 90 milliamperes. There are, however, several disadvantages: the pressure head is heavy, difficult to handle and easily damaged. Also, the life of the potentiometer is limited to only about six months operations (See Figure VI E-12).

(2) Mark V Shore Wave Recorder, University of California (Reference VI E-22): The transducer of the Mark V recorder comprises a 32 junction thermopile installed in a gas-filled rubber bellows. The reference junctions are in contact with the sea through the pressure head case, while the active junctions are acted upon by the gas. The pressure fluctuations act upon the bellows to produce temperature fluctuations in the gas which generate voltages through the thermopile. Because of the relatively short thermal time constant of the pressure head, long period pressure variations such as tides are not registered by the gage. The shore installation connected to this pressure head is a Leeds and Northrup recording millivoltimeter.

The chief advantage of this system is its inexpensive components. The pressure head is simple in design, inexpensive, rugged and easily handled; the shore installation consists of only a standard recorder. The chief disadvantage of the Mark V is the difficulty and expense of calibrating the system. Also, calibration of the instrument is not independent of wave period, depth of installation, and short period temperature fluctuations. Still another disadvantage is that gradual leakage of moisture into the rubber bellows cuts the life expectancy of the pressure head to about three months on the average. (See Figure VI E-13)

(3) Mark VI Shore Wave Recorder: Work on this recorder was prompted by the need for a high frequency response instrument to record subsurface pressure fluctuations in the surf zone. High frequency response was achieved by using; a Brush Strain Gage Recorder with flat frequency response to 100 cycles per second; and an underwater pressure head with a natural frequency that is correspondingly high.

The Mark VI Shore Wave Recorder (Figure VI E-17,) includes:

(i) a strain gage diaphragm pressure head

(ii) a four conductor waterproof cable

(iii) a Brush Strain Gage Pen Recorder

The pressure head consists of an air-backed brass diaphragm (to which strain gages are attached), an air chamber, and a water-tight cable connection.

Elastic deformation of the diaphragm due to the water pressure is measured by 4 strain gages attached to the air-backed surface of the diaphragm, 2 near the rim and 2 near the center. A continuous record is obtained by connecting the strain gage to a Brush Strain Gage Recorder. Unbalance of the strain gage bridge due to the static depth of the water can be neutralized by rebalancing the recorder after the pressure head is installed. The system is then an effective differential pressure recorder which responds to pressure fluctuations generated by the waves.

The strain gage system is not satisfactory as a total pressure recorder, (even though the diaphragm is strained according to the total pressure) because of the instability of the recorder's zero setting. Similarly, the recorder cannot be used to record such long period pressure variations as tides, because of the zero drift.

(4) Mark VIII Shore Wave Recorder, Scripps Institution of Oceanography. This recorder is a differential meter which utilizes axial strain wires to sense the differential pressure between an exposed chamber and a compliant chamber. The pressures in the compliant chamber stabilize at the average pressure by the flow of liquid silicone through a capillary. (Reference VI E-21) The Mark VIII is independent of response from the effects of - - -

- depth of installation (to 1,000 feet)
- mean temperature
- wave period, and
- suspended matter in the water.

The Mark VIII is relatively light, stable, rugged, and is simple to construct and assemble. Lowpass characteristics are easily adjusted. If desired, a flat response from low acoustic frequencies to trans-tidal frequencies is obtainable. The vacuum compliance model is expected to show no effect from abrupt water temperature changes.

This meter has the disadvantage of requiring a 4 conductor cable. However, for short-term installations, a bridge supply may be included and a 2 conductor cable employed.

The instrument is housed in a case 3 inches in diameter, 12 inches long. (Figure IV E-14). The case is constructed of commercially available seamless brass tube. The components are assembled as a series of metal disks, sealed with o-ring seals and positioned by spacers. (Figure VI E-15) The electrical seal to the cable consists of a conventional cable gland, attached to a plate which contains appropriate glass seals.

The measuring unit - a Statham strain gage - is especially adapted for use in this pressure head by the manufacturers. The Statham Laboratories Inc. of Beverly Hills, California.

The strain gages are attached to the lower diaphragm between the compliant and exposed chambers and comprise a full bridge, with four sets of axially loaded wires (Figure VI E-15).

The exposed chamber consists of a rubber bellows which is externally exposed to sea water through a perforated cap. The chamber is filled with a silicone (DC 200, of 100 centistoke viscosity). A capillary tube leads to the compliant chamber, which also contains some of the same fluid. This arrangement permits an upper cut-off of response. The entire internal mechanism is bathed in silicone.

The calibration has shown a straight-line relationship of pressure to output over the working range. The signal output of 0.2 millivolts per foot of water per 1.5 volts on the bridge. A total of 15 volts can be used on the bridge, resulting in a response of 10 chart inches per foot of pressure on the standard 2 millivolt Speedomax, and a least reading of 0.005 feet of water.

An installation in 45 feet of water has been used 1,200 feet seaward of the Scripps Institution pier. The wave record has been entirely satisfactory except for some sudden transient fluctuations. These fluctuations have been correlated with abrupt changes in water temperature occurring at the level of the instrument at the same position as the Mark VIII.

(5) Mark IX Shore Wave Recorder, (Reference VI E-48) :
The Mark IX system (Figure VI E-18) is an improvement of the Mark III potentiometer-type gage, and is designed as a general purpose instrument for permanent installations.

Its principal component is the Bourns Differential Pressure Potentiometer which is used as the unit transducer (Figure VI E-19). The movement of the pressure-sensitive brass bellows is magnified by a potentiometer-contact lever which (in the normal position of zero differential pressure) divides the resistance of the potentiometer windings equally. Variations of differential pressure cause the potentiometer contact arm to move across the potentiometer windings. The position variation of the potentiometer arm is converted to a proportional current by the bridge circuit and is recorded by the recording milliammeter.

When the instrument is submerged in a given depth of water, the static pressure at this level will act on the rubber bellows (Figure VI E-20) clamped on the shoulder provided on the bottom of the mounting flange. The rubber bellows are filled with a silicone fluid (DC 200) and this fluid is forced through two inlet pipes to the potentiometer chamber.

The first of these pipes, the Static Inlet pipe, is fitted with a sintered glass disk that restricts the flow of the silicone fluid, thus providing a "slow leak". The fluid flows through this restricted opening until the pressure within the chamber is equal to the pressure of the static level. The potentiometer case has openings at both the top and bottom to admit static pressure to the outside of the brass bellows.

An air pocket is trapped in the potentiometer chamber, against which the bellows expands and compresses as the pressure fluctuates within it. An average pressure is therefore applied to the outside of the bellows which is equal to the average static level of the instrument.

The second pipe, the Dynamic Inlet pipe, is connected directly to the inside of the potentiometer bellows. There is no impediment to the fluid flow and the dynamic pressure, caused by the wave action, acts on the inside of the brass bellows. The dynamic pressure, which varies about the average static pressure acting on the outside of the bellows, causes the movement of the brass bellows.

(6) Woods Hole Shore Wave Recorder: Developed by the Woods Hole Oceanographic Institution, (Reference IV E-26) the Woods Hole Recorder has a transducer consisting of a coil and magnet (component part of a radio speaker) arranged so that the total magnetic flux linkage of the coil varies as the water pressure fluctuates. The coil is attached to a metal bellows, the length of which varies with the pressure fluctuations. The coil is thereby moved in and out of the magnet, changing the flux linkage.

Recordings of the flux linkage of the coil can be made with a General Electric photo-electric recorder (a direct writing servo-mechanism type recorder); or with the Woods Hole photographic recorder. The Woods Hole recorder provides a record that can be used directly with the frequency analyzer developed at Woods Hole.

(7) Knapp Bottom Pressure Gage: A bottom pressure gage designed for studying harbor surging was developed and tested at the U. S. Naval Station at Long Beach, California (Reference VI E-27). A Statham strain gage unit, used in connection with a pressure - sensitive bellows, comprises the transducer of the pressure head. Four strain gages in the Statham unit are connected to form a bridge circuit which is attached to the recorder by an electrical cable. A d-c voltage is applied to the bridge, and the record is obtained by photographically recording the unbalance current with a magnetic oscillograph. Any standard strain gage recorder could be used in place of the recording system.

This gage differs from other pressure gages in that no slow leak is provided to eliminate tide and long period waves. In place of the slow leak, a solenoid valve is installed which is held open while the instrument is being raised or lowered to prevent damaging the pressure-sensing element. Once the instrument is in place, the valve is electrically closed, which seals the reference chamber at an average pressure corresponding to the depth of the water. (See Figure VI E-21)

A bottom pressure gage designed for studying harbor surging was developed and tested at the U. S. Naval Station at Long Beach California (Knapp 1951). A Statham strain gage unit used in connection with a pressure sensitive bellows comprises the transducer of the pressure head. Four strain gages in the Statham unit are connected to form a bridge circuit which is attached to the recorder by an electrical cable. A d-c voltage is applied to the bridge and the record is obtained by photographically recording the unbalance current with a magnetic oscillograph. Any standard strain gage recorder could be used in place of this recording system.

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3. Deep Water Wave Measurement:

Methods of measuring deep ocean waves are important to the oceanographer, especially where the water is too deep to anchor or to use a wave measuring device that rests on the bottom. (Reference VI E-10).

To date, information has been scanty on waves in the generating area, and also on waves along the decay path to the shore. The University of California has designed and tested several floating systems for use in very deep water. (Figures VI E-22 and 23).

The following pages describe some of the University of California's experimental work.

a. Long Line Damped Spar Buoy:

This system employed a floating spar buoy attached to a damping disk by a long wire cable. The cable was 600 feet long, which enabled the damping disk to act in water that was relatively undisturbed by wave action. The variation of buoyancy as a wave passed the buoy caused only a negligible vertical motion of the system. Thus, a reference was established for comparison with the wave heights.

b. Short Line Damped Spar Buoy:

This short-line system was developed because of the difficulty of installing and retrieving the long line system. Since the damping disk was acted upon by water moving with the surface waves, there was an appreciable vertical motion of the shore line system. This motion was accounted for by calculating the vertical motion. In these calculations, the motion of the buoy was assumed to be: the sum of the vertical components of the water particle orbits at the damping disk, plus the motion due to the buoyancy variation as the wave passed the spar buoy.

The damping disk was placed from 50 to 250 feet below water surface. When sufficient tension was maintained in the line under all conditions, the spar buoy, damping disk, and line tended to oscillate as a rigid system. When a wave passed over the system, the displacement of the spar buoy increased and a buoyancy force occurred, which tended to lift the whole system. At the same time, the damping disk also tended to move upward - due to the orbital motion of the water particles at these depths. Due to the buoyancy force of the spar buoy, the damping disk tended to move with respect to the moving water around it. This relative movement was determined through a consideration of the buoyancy force and acceleration of the system throughout a time cycle.

The effective mass of the system was much greater than that normally determined in air, since the damping disk moved a large mass of water when it moved itself.

c. Correction for the Motion of the Spar Buoy:

Equations have been derived to calculate surface wave height if:

(i) The apparent wave height is known (as found by observing the time history of the water surface on the spar buoy), or if:

(ii) the subsurface pressure time history is known (as measured by a pressure recording instrument attached to the bottom of the spar buoy)

These equations account for:

- -the orbital particle motion at the damping disk.
- -the motion of the disk relative to the particle motion
- -the hydrodynamic attenuation of pressure
- -the effect of waves passing over the spar buoy, and
- -the phase lag of the spar buoy motion relative to the surface wave.

If $H'$ is the observed wave height relative to the spar, the actual wave height is:

$$H = H'/K$$

Where:

$$K = \left[(1 - K_2 - K_3 \cos\varphi 2)^2 + (K_3 \sin\varphi)^2\right]^{\frac{1}{2}}$$

$$\tan\varphi = \frac{2\omega l}{m - \omega^2}$$

$$K_2 = e^{\frac{-2\pi D'}{L_o}}$$

$$K_3 = \frac{m(1-K_2)}{\left[(m-\omega^2)^2 + 4\omega^2 l^2\right]^{\frac{1}{2}}}$$

$$\omega = \frac{2\pi}{T}$$

$$l = \frac{C_D R^2 \rho h}{3(M_i + M_w)}$$

$$m = \frac{\rho g \pi r^2}{M_i + M_w}$$

where:

h = wave height, feet
$C_D$ = coefficient of drag
T = wave period, seconds
$L_o$ = deep water wave length
D' = depth of damping disk
r = radius of spar bucy
R = radius of damping disk
$M_i$ = mass of system parts
$M_w$ = 4 $R^3\rho$ = virtual mass of damping disk

For waves of small height and of short period, this formula can be simplified by omitting the effect of the phase angle, . In this case the equations for K and $K_3$ reduce to:

$$K = 1 - K_2 + K_3 \text{ and } K_3 = \frac{m(-K_2)}{\omega^2 - m}$$

where $K_2$ is the magnitude of the orbital motion of the water equation and $K_3$ is the magnitude of the motion of the damping disk with respect to the surrounding water.

If H' is the apparent wave height as determined from a continuous recording pressure head attached to the system, then:

$$H = \frac{H'}{K_1 - K_2 - K_3}$$

where $K_1 = e^{\frac{-2D}{L_o}}$, D being the average depth of the instrument.

Studies were also made to determine the tilt of the spar buoy for various exposed lengths and for various wind velocities. A 36 foot spar buoy of aluminum tubing (3" O.D.) weighing 49.2 pounds was attached to a 3 foot damping disk on a short line. Weights were added to the system to expose the buoy to various lengths, and the tilt was measured. Calculations and experiments indicated that not more than one-third of the buoy should be exposed: winds of more than 20 mph caused excessive tilt if 10 feet or more of the buoy was exposed.

d. Analysis of Deep Water Pressure Records:

For waves in deep water, the relationship between the sub-surface pressure and the surface wave can be found by determining the limit of the

general "K factor" equation as the depth of water becomes infinite.

$$\frac{\cosh \frac{2\pi}{L_o}\left(1 - \frac{D_i}{d}\right)}{\cosh \frac{2\pi d}{L_o}} \longrightarrow e^{\frac{-2\pi D_i}{L_o}} \quad (\text{as } d \to \infty)$$

For most practical cases this condition holds true when $d > \frac{1}{2} L_o$.

How this equation is applied to pressure records that are unsystematic in time is discussed in Part VI F: "Record Analysis."

e. Wave Recorders Used with the Spar Buoy:

A pressure recording instrument was developed to provide a record of 10 minutes duration. The total length of time scale was about 30 inches, and the pressure scale was about 10 feet per inch, with a total range of 3 inches. (See Figure VI E-24). A pressure-actuated stylus made the recording by marking a prepared wax paper located on the periphery of a drum. The drum moved during operation, guided by a helical groove cut into the shaft about which rotation took place. The rate of movement was controlled by a Standard clock mechanism for which the recording drum acted as the driving weight. The rate of fall was 1 inch per turn and a total of 5 turns was made during the recording period. (Figure VI E-17). The linear motion of the stylus was controlled from a spring-loaded sylphon bellows exposed directly to variations of the pressure in the water at the level where the instrument was placed. This recorder, called the Mark I Deep Water Wave Recorder, proved to be too delicate, and the records obtained were of insufficient length to provide good sampling for statistical studies.

The Mark II Deep Water Wave Recorder was then constructed using the component parts of the Brass Box Recorder for use with the spar buoy system to provide records 3 hours in length. This Brass Box Recorder has been discussed under Section 1b "Early Wave Measuring Instruments".

f. Suspended Underwater Pressure Recorders:

Disadvantages of the damped spar buoy sustem were difficulties of installing and retrieving the long line and large component parts. In order to overcome these difficulties, Rauch and Folsom (Reference VI E-35) suggested a scheme for suspending a pressure recorder from a float that would follow the sea surface. The pressure at the suspended recorder would vary as the recorder was raised and lowered in the water by the surface float. If the recorder was suspended more than one-half a wave length below the surface, the pressure fluctuation at the recorder would be equal to the water head through which it was raised. If suspended less than one-half a wave length below the surface, the subsurface pressure fluctuations due to wave actions would have to be considered.

The submergence of the float would be kept constant and the weight of the recorder sufficient to hold the connecting line taut. The system would operate as a rigid unit. The proper depth for the pressure recording unit would be a compromise since records independent of subsurface pressure fluctuations required great depths (800 feet or greater for wave period of about 16 seconds), but convenience of handling required as shallow a depth as possible.

To determine the pressure response for various wave periods, a theoretical investigation was conducted of the dynamics of the system. The response of the recorder was shown to be a function of the depth of the pressure recorder and the period of the wave. If the reading of the depth of the pressure recorder indicated a wave height of H' feet (Total pressure change at the instrument) the actual wave height would be: (see Figure VI E-25)

$$H = \frac{H'}{K}$$

where

$$K = 1 - \left[\frac{1}{2}\right]\left[e^{-1.225\ \frac{(d+H/2)}{T^2}} + e^{-1.225\frac{(D-H/2)}{T^2}}\right]$$

when

$D \gg \frac{H}{2}$ the expression for K reduces to

$$K = 1 - e^{\frac{-1.225D}{T^2}}$$

where

D = mean depth of instrument
T = wave period
H = wave height

g. Sonic Radio-Link Spar Buoy Wave Height Meter (Reference VI E-5)

Two types of radio-linked spar buoy systems were studied at the U. S. Navy Radio and Sound Laboratories during 1945; (1) a radio-linked buoy that measures the wave height by sonar means, and (2) a radio-linked buoy that measures the wave height by a pressure-actuated device. The radio-link was provided to eliminate the need of recovering the instrument to obtain the record. A sonar-type unit was developed and several field experiments were made with this unit. No pressure actuated type instrument was developed at that time.

A 25-foot spar buoy was made of 4 inch O.D. dural tubing, with the transmitter mounted at one end and the sonic transducer at the other (Figure VI E-26). The transducer was mounted on a bracket at the bottom of the buoy, facing up toward the surface of the water. Pulses of sound energy were radiated from the transducer to obtain the echoes from the surface of the water and also the echoes from two reflectors fastened 5 feet apart along the side of the buoy. The transmitted signal consisted of an initiating pulse; two echoes from reflector pads a known distance apart; and an echo from the water surface. The receiver signal was amplified and applied to the unblanking circuit of a synchronized oscillograph. The oscilloscope was then photographed with an oscillographic camera, which converted the spot presentation of the oscilloscope into a continuous strip record, (Figure VI E-27).

This system was successfully used on several occasions. Comparison of these readings to moving pictures taken of the waves passing the spar buoy, showed the error to be less than 0.2 feet. Although comparatively satisfactory, this sonic radio-link method of measuring ocean waves has not been

further developed since those first tests..

4. Measurement of Long Period Waves:

Waves in a storm area form a most irregular pattern, and have a wide range of frequencies. Under storm conditions, most of the energy of the waves is concentrated, however, within a relatively narrow range of periods, say from 5 to 9 seconds. After the waves leave the storm area, they travel as swells. Selective attenuation of the shorter periods leads to a gradual shift of the energy maximum toward the longer periods. The most prominent swell from a storm area several thousand miles away has a period of 12 to 16 seconds, and an amplitude of about 2 feet or less. The longest swell period visually observed is 22 seconds.

During the last few years, a number of automatic swell-recording instruments have been installed by British and American organizations. These instruments have demonstrated the existence of low long forerunners to the swell, with periods up to about 30 seconds. The semidiurnal and diurnal tides are 12 octaves lower on the frequency scale than swells.

Four other types of waves have been noticed whose periods are between the wind generated swell and the tides. These are:

(i) Tsunamis - caused by submarine earthquakes or eruptions. (Popularly known as tidal waves, but they have no relation to tide-producing forces.) (Reference VI E-32)

(ii) Seiches - caused by atmospheric variations (Reference VI E-32)

(iii) Surf beats - related to the fluctuation in height of the incoming wind-generated waves. (Reference VI E-31)

(iv) Harbor surging - the oscillation found in harbors (Reference VI E-27)

a. Deep Water Mark III Wave Recorder:

Hydrodynamic theory shows that pressure-type instruments installed in water about 600 feet deep will record the long period wave but not the wind generated waves of 4 to 20 seconds period. The pressure recorder must be installed in depths less than one-half the wave length of the shortest wave to be recorded to experience more than one percent of the total pressure fluctuation generated by the wave at the surface of the water. Thus, recorders installed in deep water will be able to feel pressure variations of only the long period waves and tides.

In 1947, the Mark III, Model 3, deep water shore wave recorder, shortly after its construction by the University of California in 1947, was tested in 600 feet of water at Drakes Bay and near the Farallon Islands. There was considerable difficulty in making the installations, but records were obtained at both places. Tide effects were filtered from the record by a slow leak in the static pressure chamber.

The records obtained did not show pressure variations that could be related to surface waves. The pressure fluctuations were random without any evident periodicity. Frequently readings remained above, or below, the centerline of the chart (indicating an increase or decrease in pressure respectively) for as long as 15 minutes. The sequence of these readings was erratic and it seemed impossible that these changes in pressure could be caused by waves.

The explanation given for these records was that no surface waves of sufficient length and height to be registered by the pressure head occurred during the tests. The random variations were probably due to pressure changes in the instrument's air dome, caused by temperature fluctuations. Temperature changes of only one degree, or less, can cause the sort of variations recorded by this instrument, and temperature fluctuations (temperature waves) are known to exist in these depths.

The Mark III, Model 3, Pressure Recorder (Reference VI E-7) is identical in design to the Mark III, Model 2, Shore Wave Recorder, discussed under "Present Day Instruments" (Section 2c), except that it has a larger air dome and a full scale reading for differential pressures of ± 2 feet of sea water. If the pressure head is installed in 600 feet of water, the absolute pressure in the air dome would be 632 feet of water. For a temperature change of one degree centigrade with a water temperature of 10 degrees centigrade, the change in pressure in the air dome would be:

$$\Delta p = 632 - 632\left[\frac{283 \pm 1}{283}\right] = \pm \left[\frac{632}{283}\right] = \pm 2.24 \text{ feet of water}$$

b. Tsunami Recorder - Scripps Institution of Oceanography:

The Scripps recorders, illustrated in Figure VI E-22, were installed in shallow water, where pressure fluctuations of all wave periods and the tides were present. Both the short period wind wave and tide pressure fluctuations were removed by means of a tuned hydraulic filter and the remaining long period waves were recorded by a pen recorder.

The Mark I (2 stage) tsunami recorder operated at the end of the Scripps pier from January to August, 1948. This instrument (which is sensitive to waves with frequencies lying between those of the wind-generated swell and the astronomic tides) revealed, during times of high wind wave, the presence of irregular oscillations of several minutes period. These waves were called "surf beat", as they are related to fluctuations in the height of incoming waves.

Waves of 15 - to 30 - minute periods were also observed and tentatively correlated with meteorological distrubances. These storm "seiches" were largely obscured by the relatively high surf beat. Accordingly, it was decided to retune the circuit to lower frequencies, shifting the period of peak response from 15 to 45 minutes (Mark II, 3 stage).

Two Mark II instruments (Reference VI E-37) have been installed, one on the end of the Scripps Institution pier and the other at the end of the municipal pier at Oceanside, California. They differ in minor details of construction, but are designed for identical responses. The purpose of the dual installation is to provide information as to the direction of wave approach.

5. Measurement of Wave Direction:

Wave direction can be determined from aerial photographs only if suitable lighting conditions exist, and if the water surface is not confused by local wind chop. Visual observation of the sea from any vantage point will enable an approximate determination of wave direction, but again local wind chop may confuse the surface. The placement of three pressure recorders in a triangle, and timing the arrival of the waves at the various recorders has been suggested as a possible method of measuring wave direction. Difficulty will probably arise if this system is attempted, because of the short-crested characteristics of the waves outside the crest zone. The crest length of the waves tends to be about the same as the wave length. Therefore, it is possible that a wave might act on two of the recorders and not on the other.

a. Rayleigh Disk:

No instruments have been built to date which will measure wave direction. The Rayleigh Disk (see Figure VI E-30), which was considered a possible solution to this measurement problem, has been investigated but proved unsatisfactory both from an experimental and theoretical standpoint. A study of the behavior of this disk under the action of two or more wave systems was recently completed by the Beach Erosion Board. Their conclusions were summarized in Technical Memorandum No. 18 (Reference VI E-3) as follows:

> "It appears that the records made under natural conditions confirm as nearly as might be expected the theoretical analysis of the behavior of the disk when acted upon by two sine wave systems.
>
> It also appears from the study that the presence of two or more wave systems, the presence of strong currents other than wave currents such as rip tide, and the uncertainty in analysis of the records proved the Rayleigh Disk to be unreliable as a wave direction indicator."

b. An Electromagnetic Current Meter:

An electromagnetic method of measuring water currents has been studied as a way to measure wave direction. This system was based on a suggestion made by Faraday in the Bakerian Lecture of 1832 before the Royal Society (Reference VI E-51)

> "Theoretically, it seems a necessary consequence that, where water is flowing, there electric currents should be formed: thus, if a line be imagined passing from Dover to Calais through the sea, and returning through the land beneath the water to Dover, it traces out a circuit of conducting matter, one part of which, when the water moves up or down the channel, is cutting magnetic curves of the earth, whilst the other is relatively at rest."

Faraday attempted to detect electric currents flowing over a short range response to the water motions of the Thames, but the potentials developed at the copper electrodes used masked the effect. However after the middle of the 19th century, the effect was noted repeatedly on the very long ranges provided by broken submarine telegraph cables.

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Scripps Institution of Oceanography has worked on a current meter (Figure VI E-30) based on the electromagnetic method. Rough measurements demonstrated that measurable voltages were present in the sea that could be closely correlated with water-particle motion accompanying wave action. The instrument had three electrodes mounted 2 feet apart and 2 feet above the corners of a heavy concrete base. It was placed on the bottom in such a way that electrodes 1 and 2 detected potentials along the direction of water motion while 2 and 3 detected potentials perpendicular to the motion. From the theory, one would expect greater potentials to be developed across the direction of water motion than along it. For some reason, this was not the case in the records obtained. The electrodes were in 20 feet of water near the end of Scripps pier. The sea was calm except for a 2-foot swell of 8 seconds period. The period of the larger fluctuations in the record was 15 to 20 seconds, but minor irregularities occurred every 8 seconds. The pattern of the major variations suggested the phenomenon of surf beat.

c. English Current Meters:

The National Institute of Oceanography (Reference VI E-33) briefly mentions current-measuring instruments being developed in England, as follows:

> "A surface-current meter has been designed by Mr. Pierce, using basic ideas described by Dr. Curruthers for the further development of his Vertical Log Current Meter. It can be moored and left unattended for 14 days and is too large and heavy to be easily influenced by drifting weed and wave movement. It is made of non-magnetic materials, and tests of the special heavily damped compass made at the Admiralty Compass Observatory show that the recorded bearing should be accurate to within a few degrees...
>
> Two other meters have been made to measure bottom currents, particularly the beach currents due to waves. One records the varying deflection of a can mounted vertically on a flexible rod: the other records the varying e.m.f.'s induced by the flow of water through an artificial magnetic field. Both instruments have been calibrated in the tank...
>
> A fourth instrument with magnetically coupled pressure-plate propeller and photographic recording is being constructed. It is much lighter than the others and is designed so that a number of them can be lowered one below the other, from an anchored ship or buoy..."

d. Knapp Current Meter:

A current meter designed for studying harbor surging was developed and tested at the U. S. Naval Station at Long Beach, California (Reference VI E-27). A strain gage system measures the force exerted on a sphere exposed to the underwater current; and a commutator with a ring contact attached to the sphere indicates the direction of the underwater current. A d-c voltage is applied to the strain gages and the commutator ring so that records of the current velocity and direction can be obtained on a magnetic oscillograph.

6. References:

VI E-1: Admiralty Research Laboratory (1947),
The Generation and Propagation of Ocean Waves and Swell,
Teddington, Middlesex, England,
Feb. 27, 1947.

VI E-2: Beach Erosion Board (1948),
An Ocean Wave Measuring Instrument,
Technical Memorandum No. 6, Corps of Engineers,
October, 1948.

VI E-3: Beach Erosion Board (1950),
The Rayleigh Disk as a Wave Direction Indicator,
Technical Memorandum No. 18, Corps of Engineers,
July, 1950.

VI E-4: Burns, E. W. and W. W. Kuanetzow
A Hydrostatic Wave Recorder,
Governmental Hydrographic Institute of Leningrad,
Icelandic Seas
(Grossudarstvenniye Gidrographicheskiye Institute, Isslandovaniye Moriye)
USSR, Nos. 20-25, 1934-37, Fasc. 24
(in Russian, German summary)

VI E-5: Bureau of Ships, Interim Report 1948
"Sonic Radio-Link Wave Height Meter",
U. S. Navy Radio and Sound Laboratories, Problem U24CD

VI E-6: Chatley, Herbert (1939),
The Measurement of Tides and Waves by Echo-Sounders,
The Dock and Harbor Authority,
Dec. 1939.

VI E-7: Chinn, Allen J. (1949),
Summary Report on Shore Wave Recorder Mark III,
Technical Report HE-116-303
University of California, Institute of Engineering Research, Berkeley,
June 21, 1949.

VI E-8: Chinn, Allen J. and J. D. Isaacs, (1948),
The Testing and Calibration of the Mark III Shore Wave Recorders (Manufactured by the Mare Island Navy Yard),
Tech. Report #HE-152-1,
U. C. Institute of Engineering Research Berkeley,
June 16, 1948.

VI E-9: Folsom, R. G. (1944),
October Tests at Scripps Institution of Oceanography:
Anchored Float with Graduated Mast,
Tech. Report HE-116-24,
U. C. Institute of Engineering Research, Berkeley,
November 11, 1944.

VI E-10: Folsom, R. G. (1945a)
Wave Measurements in Deep Water with Recording Instruments,
Tech. Report HE-116-90,
U. C. Institute of Engineering Research, Berkeley,
Feb. 1, 1945.

VI E-11: Folsom, R. G. (1945b),
Wave Height Determinations from Smoke Photographs at Sea,
Tech. Report HE-116-79,
U. C. Institute of Engineering Research, Berkeley,
April 2, 1945.

VI E-12: Folsom, R. G. (1946),
Field Test of Shore Wave Recorders,
Technical Report HE-116-180,
U. C. Institute of Engineering Research, Berkeley,
Jan. 15, 1946.

VI E-13: Folsom, R. G. (1947),
"Sub-Surface Pressures Due to Oscillatory Waves",
Transactions, American Geophysical Union,
Vol. 28, No. 6,
pp. 875-881, Dec. 1947.

VI E-14. Folsom, R. G. (1948),
Measurement of Ocean Waves,
(Technical Report HE-116-282),
U. C. Institute of Engineering Research, Berkeley,
May 15, 1948.

VI E-15: Folsom, R. G. and J. T. Westerlund (1945),
Summary of Work with a N. O. L. Acoustic System, Mark I Model 3,
(Technical Report HE-116-151),
U. C. Institute of Engineering Research, Berkeley,
July 30, 1945.

VI E-16: Gaillard, D. D. (Capt., Corps of Engineers, U. S. A.) (1904),
Wave Action in Relation to Engineering Structures,
The Engineer School, Fort Belvoir, Virginia,
1904.

VI E-17: Graybeal, T. D. (1945a),
Report on Cambridge Swell Recorder,
Tech. Report HE-116-88,
U. C. Institute of Engineering Research, Berkeley,
April 9, 1945.

VI E-18: Graybeal, T. D. (1945b),
Laboratory Calibration of Shore Wave Recorder, Mark I,
Technical Report HE-116-157,
U. C. Institute of Engineering Research, Berkeley,
Sept. 14, 1945.

VI E-19: Isaacs, J. D. (1945a),
A Device for Traversing the Surf Zone of Ocean Beaches,
Tech. Report HE-116-90,
U. C. Institute of Engineering Research, Berkeley,
March 25, 1945.

VI E-20: Isaacs, J. D. (1945b),
Revised Method of Operating the Sighting Bar Wave Meter,
Tech. Report HE-116-129,
U. C. Institute of Engineering Research, Berkeley,
July 7, 1945.

VI E-21: Isaacs, J. D. and J. Rush, Jr. (1951),
A Strain Gage Wave Meter (unpublished paper)
Scripps Institution of Oceanography,
La Jolla, California,
March 8, 1951.

VI E-22: Isaacs, J. D. and Robert L. Wiegel (1950),
"The Thermopile Wave Meter",
Transactions, American Geophysical Union, Vol. 31, No. 5,
Part I
Oct. 1950. pp. 711-716.

VI E-23: Iversen, H. W. (1950),
Sighting Bar for Observations of Breaker Height and Distance to Breakers,
Tech. Report HE-116-42,
U. C. Institute of Engineering Research, Berkeley,
Jan. 3, 1945.

VI E-24: Iversen, H. W. (1945b),
Instructions for the Operation of the Wave Recorder at Estero Bay,
Tech. Report HE-116-70,
U. C. Institute of Engineering Research, Berkeley,
March 26, 1945.

VI E-25: Johnson, J.W. (1945),
Instructions for Use of Beach Poles in Measuring Wave Heights,
Tech. Report HE-116-158,
U. C. Institute of Engineering Research, Berkeley,
Aug. 6, 1945.

VI E-26: Klebba, Arthur A. (1945),
A Summary of Shore Recording Wavemeters,
Woods Hole Oceanographic Institution
Aug. 3, 1945.

VI E-27: Knapp, Robert T.
"Determination of Wave, Surge, and Ship Motion - U. S. Naval Station, Long Beach, California",
Final Report of Contract NOy-13116,
Bureau of Yards and Docks, U. S. Navy,
May, 1951.

VI E-28: Morison, Jack R. and Frank E. Snodgrass (1949),
Unpublished Report on Mark IV Pressure Head,
U. C. Institute of Engineering Research, Berkeley,
April 11, 1949.

VI E-29: Munk, Walter H. (1947a),
"Tracking Storms by Forerunners of Swell"
The Journal of Meteorology, Vol. 4, No. 2,
April, 1947. pp. 45-57.

VI E-30: Munk, Walter H. (1947b),
"Increase in the Period of Waves Travelling over Large Distances: with Applications to Tsunamis, Swell, and Seismic Surface Waves",
Transactions, American Geophysical Union, Vol. 28, No. 2,
April, 1947, pp. 198-217.

VI E-31: Munk, Walter H. (1949),
"Surf Beats", Transactions,
American Geophysical Union, Vol. 30, No. 6,
(contribution from the Scripps Institution of Oceanography New Series No. 434),
Dec. 1949. pp. 849-854.

VI E-32: Munk, Walter H. and Hector V. Iglesias and T. R. Folsom (1948),
An Instrument for Recording Ultra-low Frequency Ocean Waves,
Scripps Institution of Oceanography, Wave Report No. 81,
Oct. 1948.

VI E-33: National Institute of Oceanography (1949-1950),
Annual Report,
Cambridge University Press,
1949-1950. p. 15.

VI E-34: Putz, Robert R. (1950),
Wave Height Variability, Series 3, Issue 318,
U. C. Institute of Engineering Research, Berkeley, (unpublished)
Sept. 1950.

VI E-35: Rauch, S. E. and R. G. Folsom (1945),
Evaluation of K for the Suspended Underwater Pressure Recorder,
Tech. Report HE-116-156,
U. C. Institute of Engineering Research, Berkeley,
Aug. 6, 1945.

VI E-36: Rudnick, Philip (1949),
A System for Recording and Analyzing Random Processes,
U. C. Marine Physical Laboratory, San Diego, Calif.,
October 5, 1949.

VI E-37: Scanlon, T. S., Jr. and P. Osborn (1950),
Mark II Tsunami Recorder,
Scripps Institution of Oceanography Wave Report No. 97,
Sept. 1, 1950.

VI E-38: Schumacher, Arnold (1932),
"Photographic Measurement of Ocean Waves"
Zeitschrift der Gesellschaft fur Erdknunde zu Berlin,
Ertanzungsheft III,
1932.

VI E-39: Scripps Institution of Oceanography (1944),
Proposed Uniform Procedure for Observing Waves and Interpreting Instrument Records,
(S. I. O. Wave Project Report No. 26,)
Dec. 12, 1944.

VI E-40: Scripps Institution of Oceanography (1950),
Quarterly Progress Report No. 4,
(Beach Erosion Board Contract W-49-055-eng-3),
April, May, June, 1950.

VI E-41: Seiwell, H. R. (1947),
"Investigation of Underwater Pressure Records and Simultaneous Sea Surface Patterns",
Transactions, American Geophysical Union, Vol. 28, No. 5,
(contribution No. 386, Woods Hole Oceanographic Institution),
Oct. 1947. pp. 722-724.

VI E-42: Seiwell, H. R. (1949a),
"Sea Surface Roughness Measurements in Theory and Practice",
Annals of the New York Academy of Science, Vol. 51, Art. 3,
(contribution No. 435 Woods Hole Oceanographic Institution,
Woods Hole Mass.),
May 13, 1949. pp. 483-501.

VI E-43: Seiwell, H. R. (1949b),
"The Principles of Time Series Analysis Applied to Ocean Wave Data",
Proceedings of the National Academy of Sciences,
Vol. 35, No. 9,
Sept. 1949. pp. 518-528.

VI E-44: Seiwell, H. R. and George P. Wadsworth (1949),
"A New Development in Ocean Wave Research",
Science,
Vol. 109, No. 2829,
March 18, 1949. pp. 271-274.

VI E-45: Shepard, Francis P. and Eugene C. LaFond (1944),
"Sand Movements along the Scripps Institution Pier",
American Journal of Science,
Vol. 238, pp. 272-285.

VI E-46: Snodgrass, Frank E. (1950),
A History of Ocean Wave Measuring Instruments,
(A Report presented to the Graduate Seminar in Electrical Engineering.
U. C. Berkeley), Fall Term,
1950.

VI E-47: Snodgrass, Frank E. and Donald E. Stiling (1951a),
Analysis of Wave Records,
U. C. Fluid Mechanics Lab., Berkeley,
Tech. Report HE-116-307,
1951.

VI E-48: Snodgrass, Frank E. and Donald E. Stiling (1951b),
Operation Manual for Mark IX Shore Wave Recording System,
Tech, Report HE-116-320,
U. C. Fluid Mechanics Lab., Berkeley,
March, 1951.

VI E-49: University of California (1945), Institute of Engineering Research,
Final Report on Wave Investigations,
Tech. Report HE-116-165, Berkeley,
(Buships Contract No. 16290)
Aug. 31, 1945.

VI E-50: Valembois, J. (1949),
Recording Manometer for Submarine Pressure,
La Houille Blanche No. Special B-1949,
Extract from The Bulletin of the Beach Erosion Board Vol. 3,
July 1, 1949.

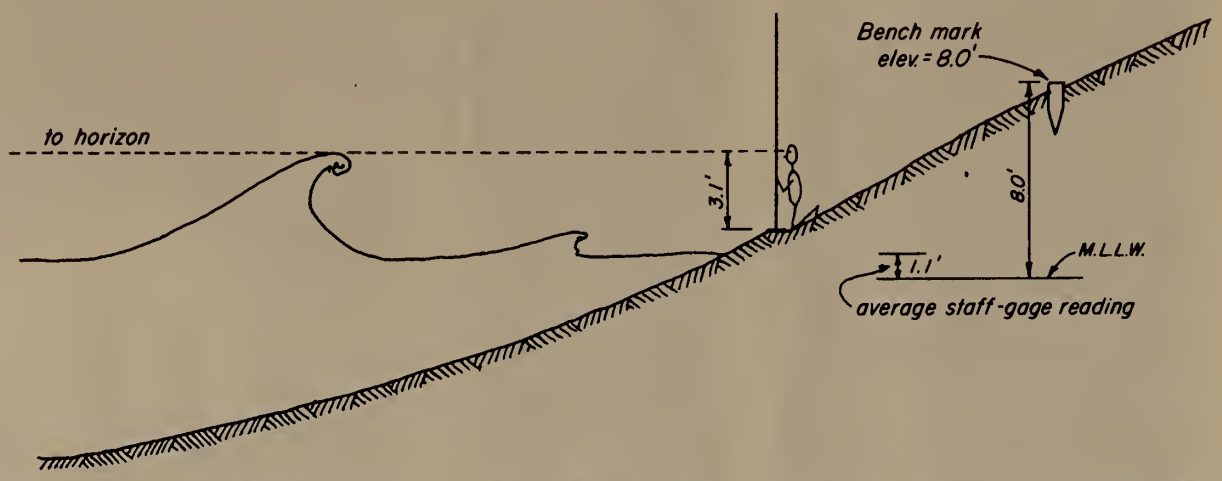
VI E-51: von Arx, W. S. (1950),
An Electromagnetic Method for Measuring the Velocities of Ocean Currents from a Ship Under Way.
Woods Hole Oceanographic Institution,
March, 1950.

VI E-52: Wiegel, Robert L. (1947a),
Tables of the Functions of d/L and d/$L_o$,
Tech. Report HE-116-265,
U. C. Institute of Engineering Research, Berkeley,
Jan. 30, 1947.

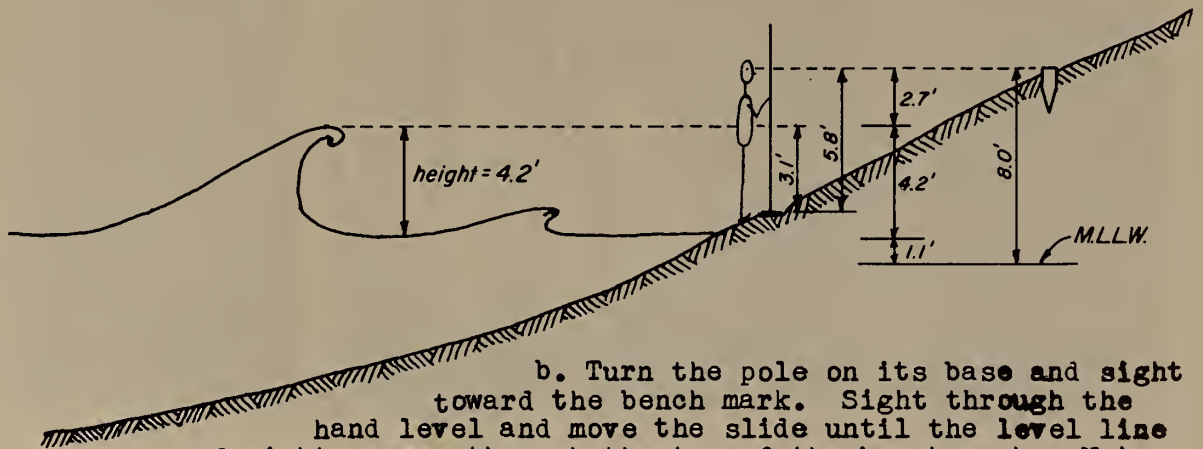
VI E-53: Wiegel, Robert L. (1947b),
Measurement of Surface Waves by Electrodes,
Tech. Report HE-116-269,
U. C. Institute of Engineering Research, Berkeley,
Sept. 30, 1947.

VI E-54: Wiegel, Robert L. (1948),
Pressure Response at the Ocean Bottom Due to Surface Gravity Waves,
(Tech. Report HE-116-108),
U. C. Institute of Engineering Research, Berkeley,
Sept. 8, 1948.

VI E-55: Wiegel, Robert L. (1949),
An Analysis of Data From Wave Recorders on the Pacific Coast of the United States,
Tech. Report HE-116-289,
U. C. Institute of Engineering Research, Berkeley,
Jan. 2, 1949.




a. Stand or squat behind the pole, sight at the horizon and move the eye up or down the pole until the crests of the majority of big waves appear to touch the horizon. Move the slide on the pole until it is at eye level. (If the horizon cannot be seen because of fog, use the hand level on the slide to obtain a horizontal sight.) Read the position of the top of the slide to the nearest tenth of a foot.
Example: reading is 3.1 feet




b. Turn the pole on its base and sight toward the bench mark. Sight through the hand level and move the slide until the level line of sight passes through the top of the bench mark. Note the reading of the top of slide to the nearest tenth of a foot.
Example: reading is 5.8 feet
Difference in elevation between crest and BM = 5.8 - 3.1 = 2.7 feet
Elevation of wave crest above MLLW = 8.0 - 2.7 = 5.3 feet
Average staff gage reading = 1.1 feet
Height of waves = 5.3 - 1.1 = 4.2 feet



INSTRUCTIONS FOR USE OF SIGHTING POLE FIGURE VI-E-1

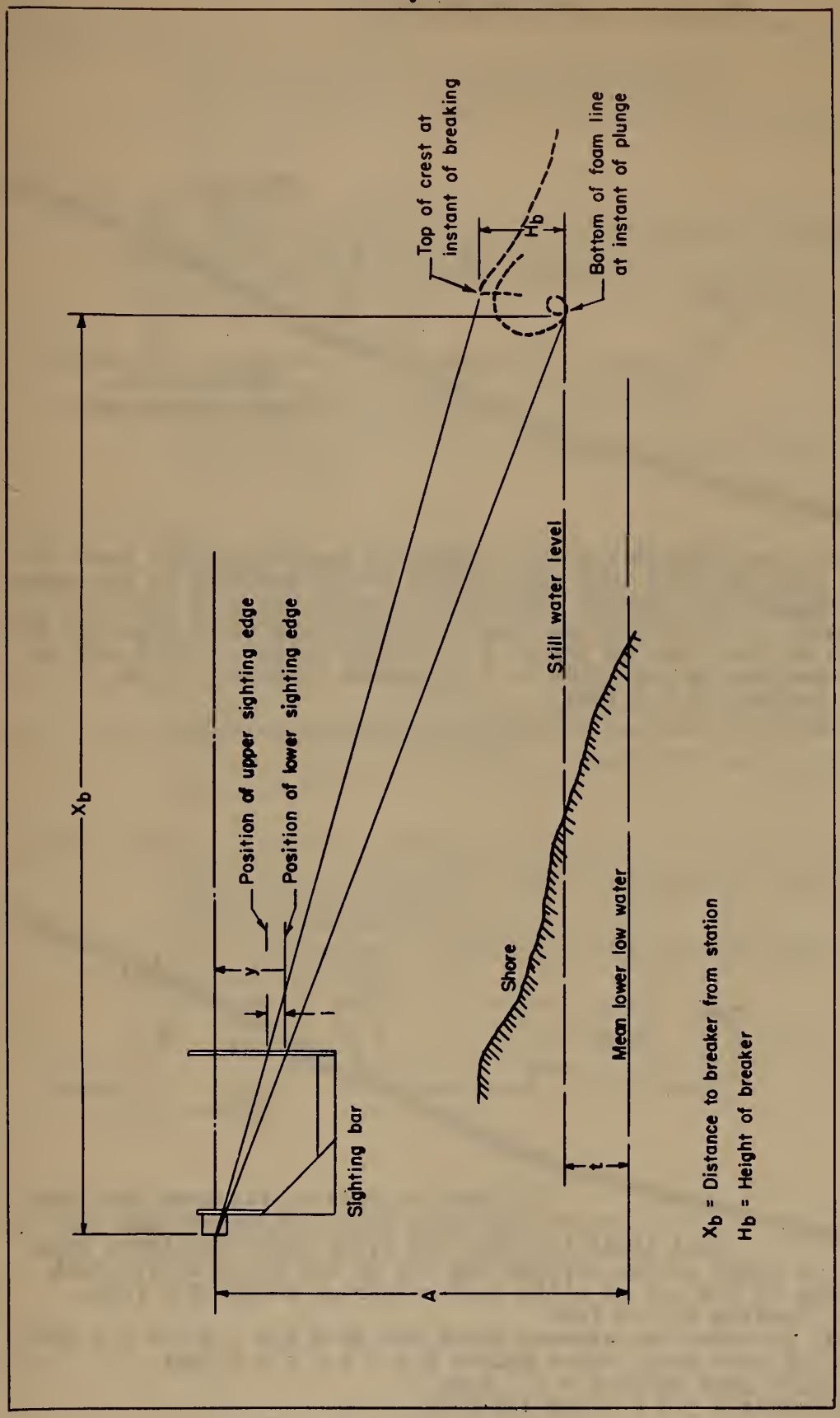
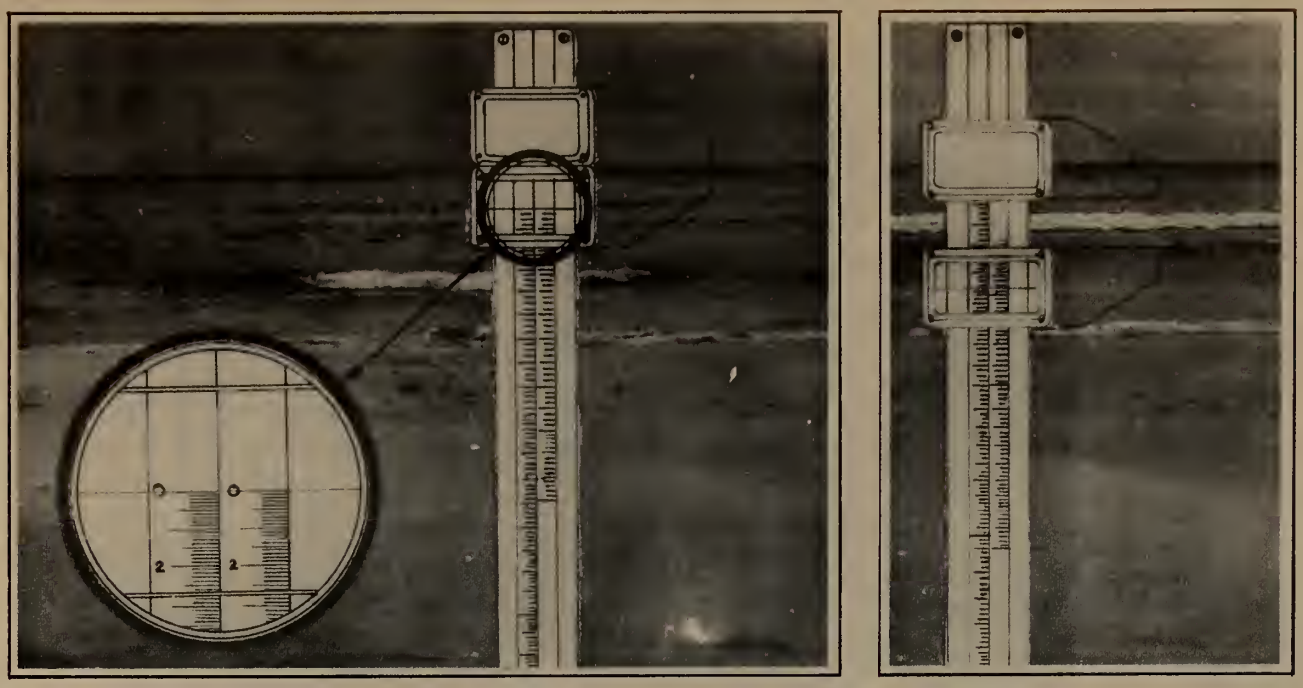
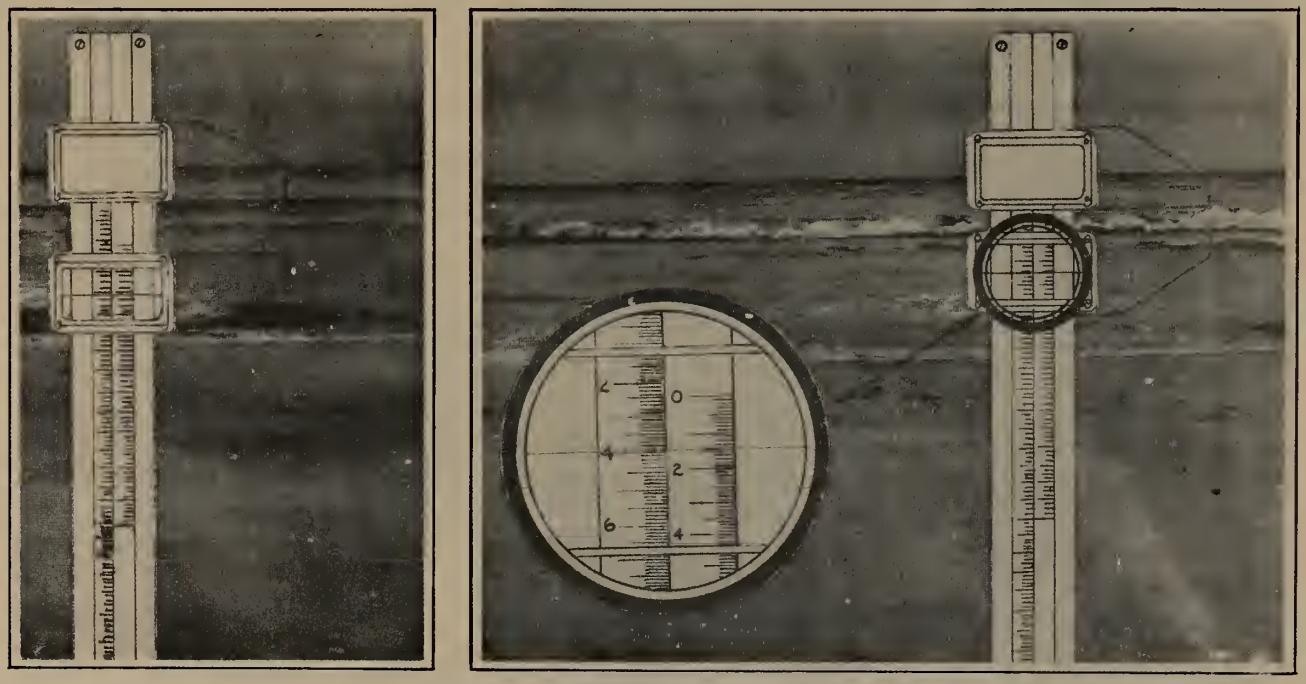



FIG. VII E-2 - GEOMETRY OF SIGHTING BAR OBSERVATIONS



a. b.



c. d.

FIG. VI E-3 - Observer's view through Sighting Bar



FIG. VIE-4 - Observer making sighting bar observation

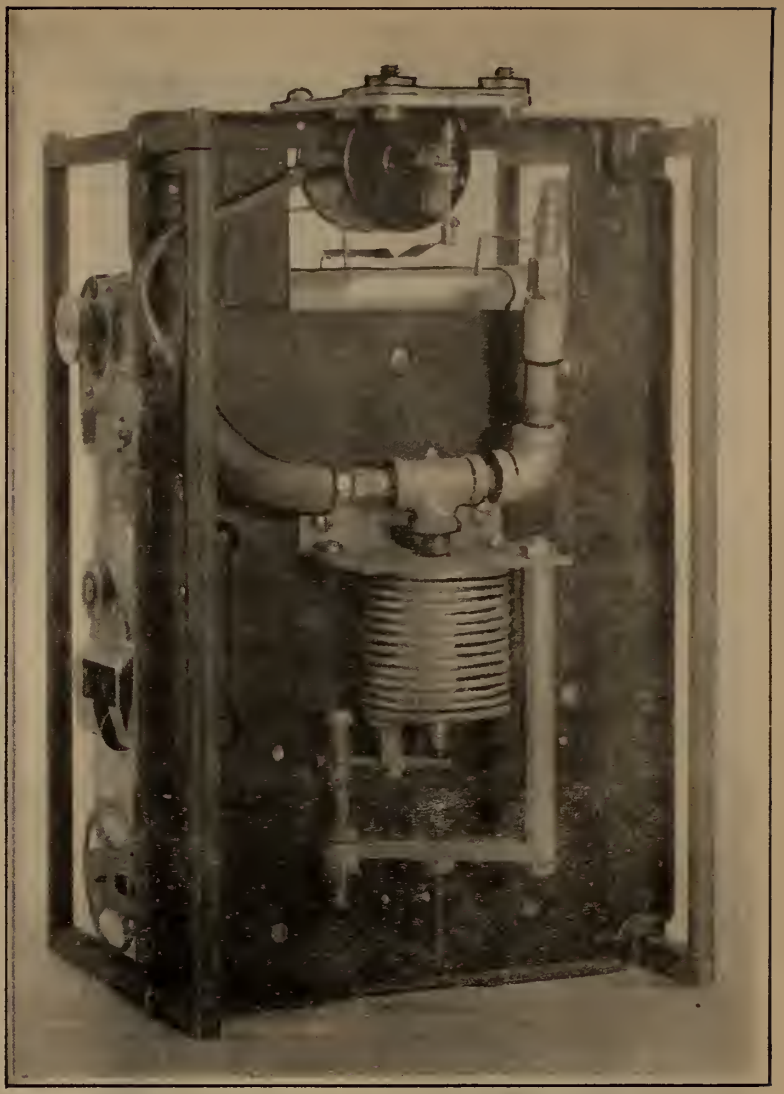


FIG. VIE-5 - Improved pressure release valve of brass box



RESTRICTED Security Information



FIG. VIE-6 - The Brass Box wave recorder adapted for deep water measurements (deep water wave recorder Mark II)

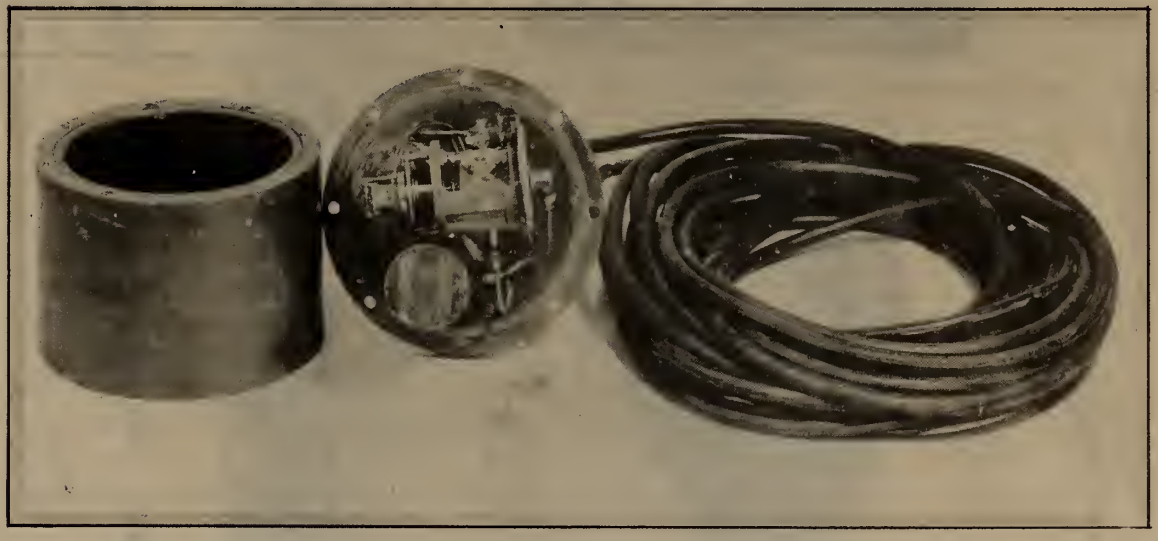


FIG. VIE-7 - Rheostat-differential pressure unit showing rheostat and differential pressure element

"CLASSIFICATION CANCELLED OR CHANGED
Confidential, BY AUTHORITY
OPNAV 26034 8 Dec 53
(SIGNATURE) M. L. Brown (RANK) Lt.
(DATE)

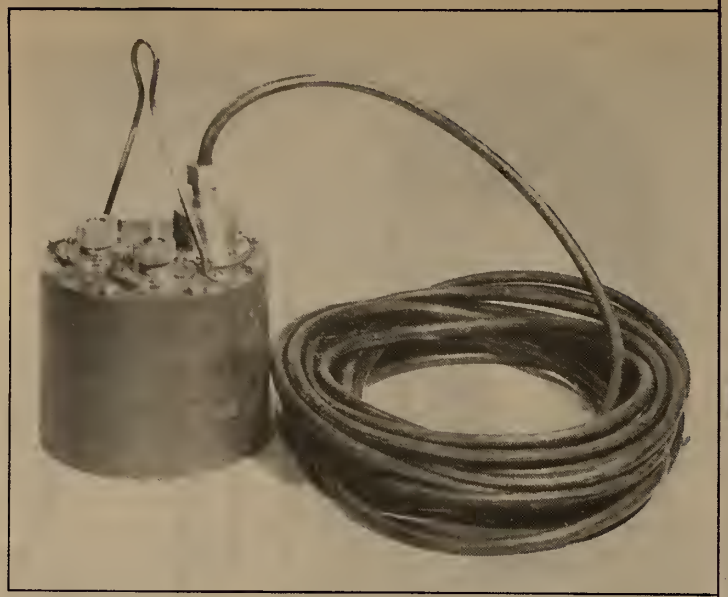


FIG. VI E-8 - Rheostat-differential pressure unit assembled




FIG. VI E-9 (upper right) - Total Pressure-Time recorder showing sylphon bellows, cantilever beam, strain gages and SR-4 recorder, Shore Wave Recorder Mark III

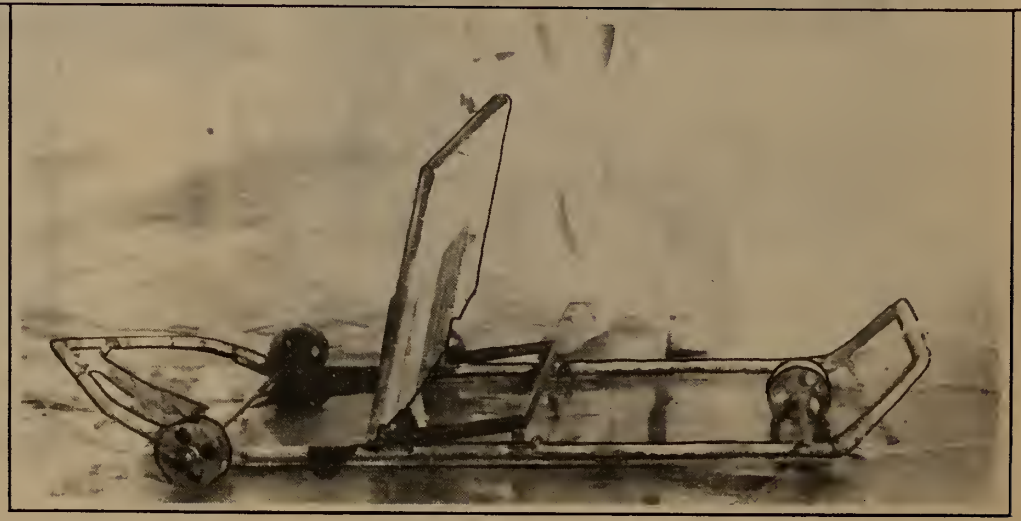


FIG. VI E-10 (right) - Sea sled with flap up



RESTRICTED Security Information



a. Installation at Huntington Beach pier, California



b. Installation at Elwood pier, California

FIG. VIE-11 - Beach Erosion Board
Step-resistance Wave Gages

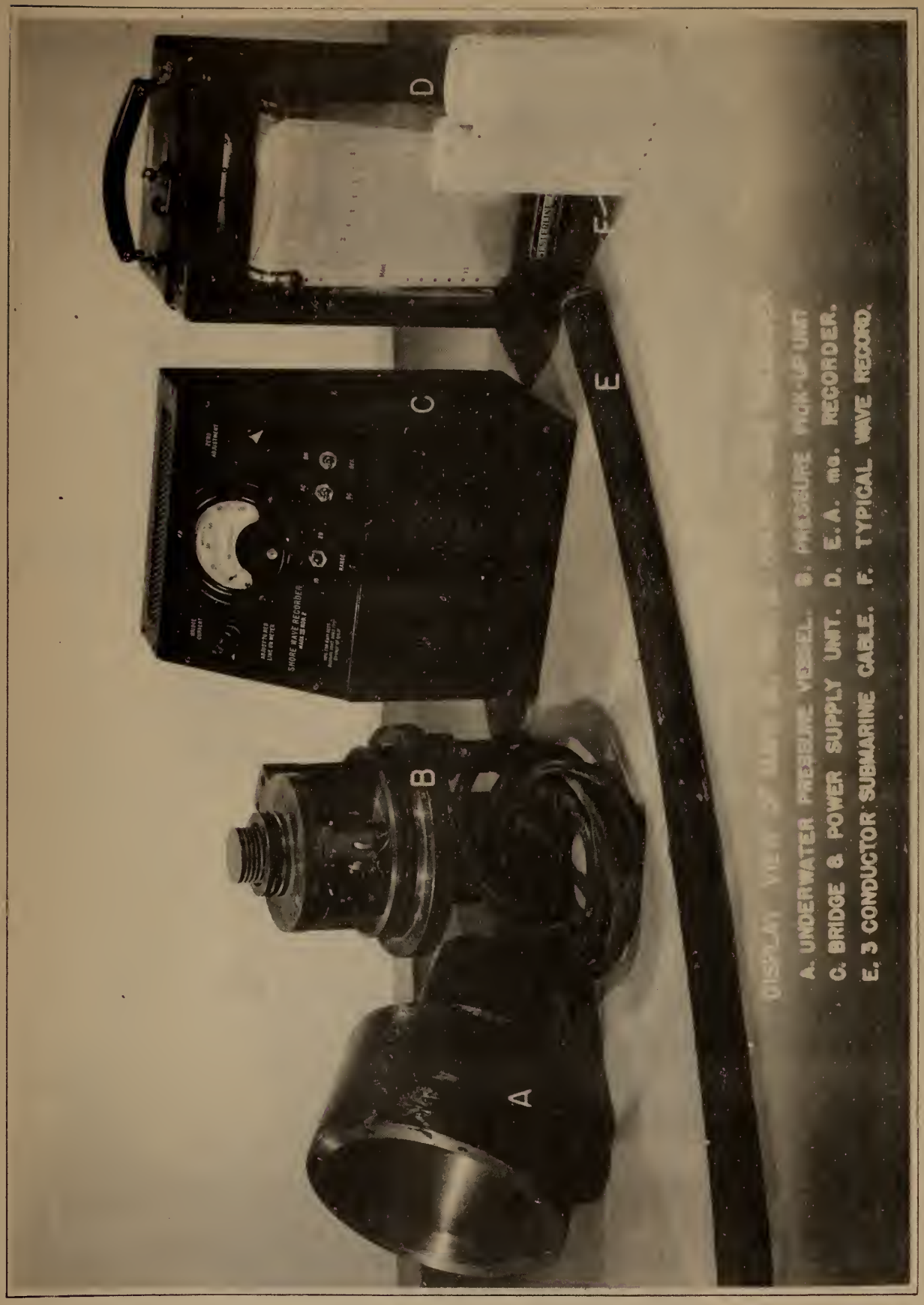



FIGURE XII E-12

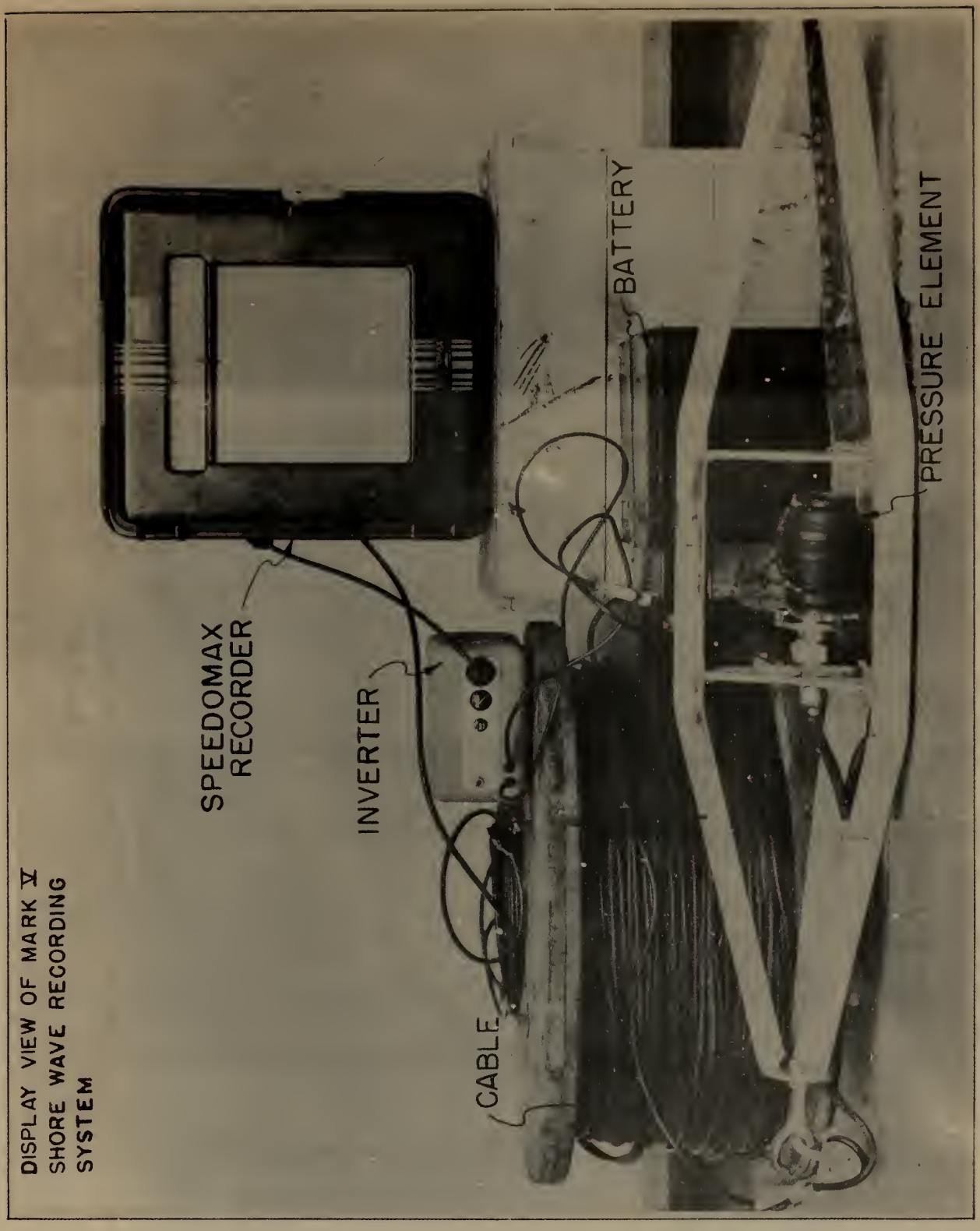
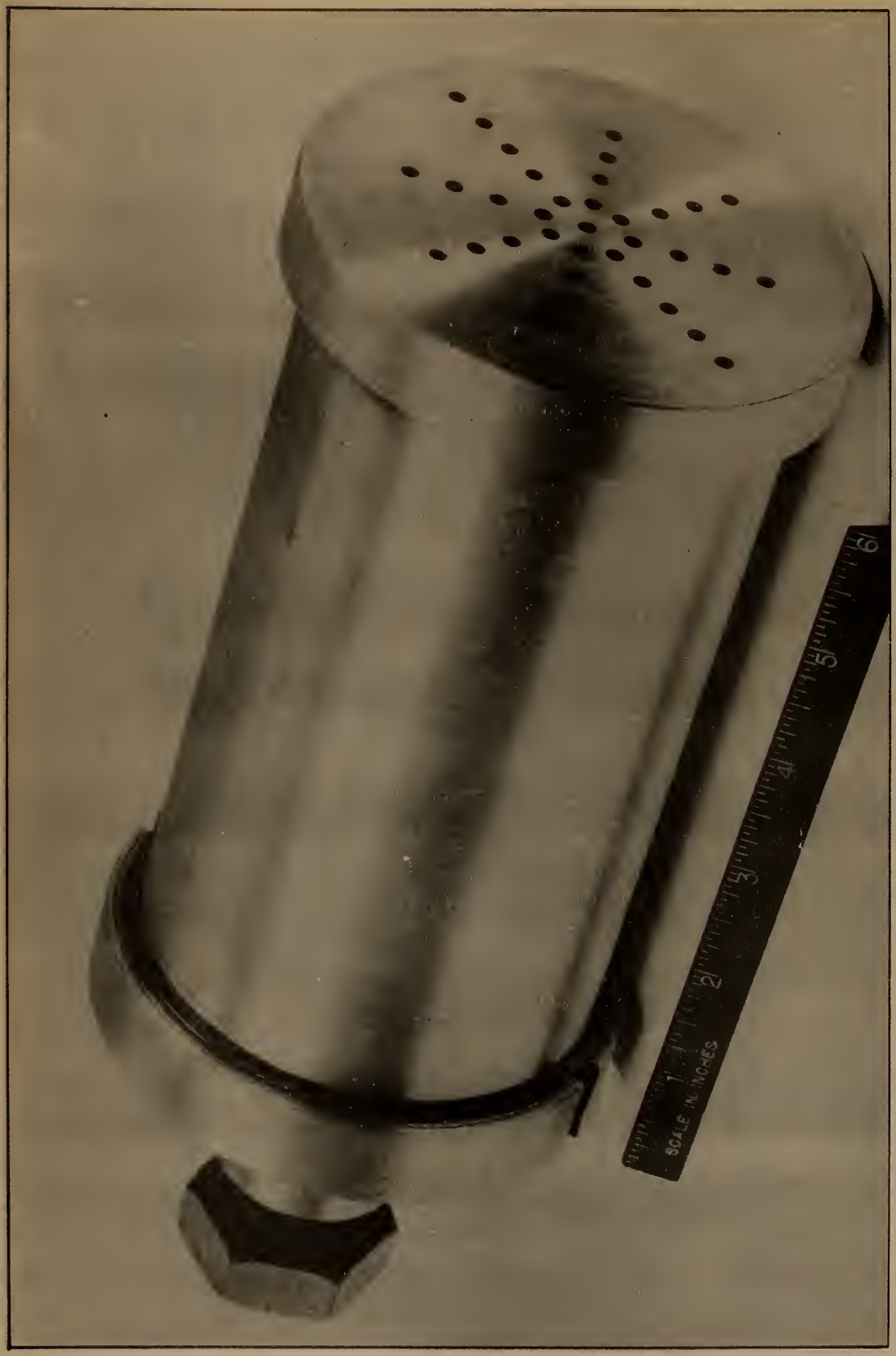



FIGURE VII E-13




SCRIPPS INSTITUTION OF OCEANOGRAPHY WAVE METER ASSEMBLY

FIG. XIE-14

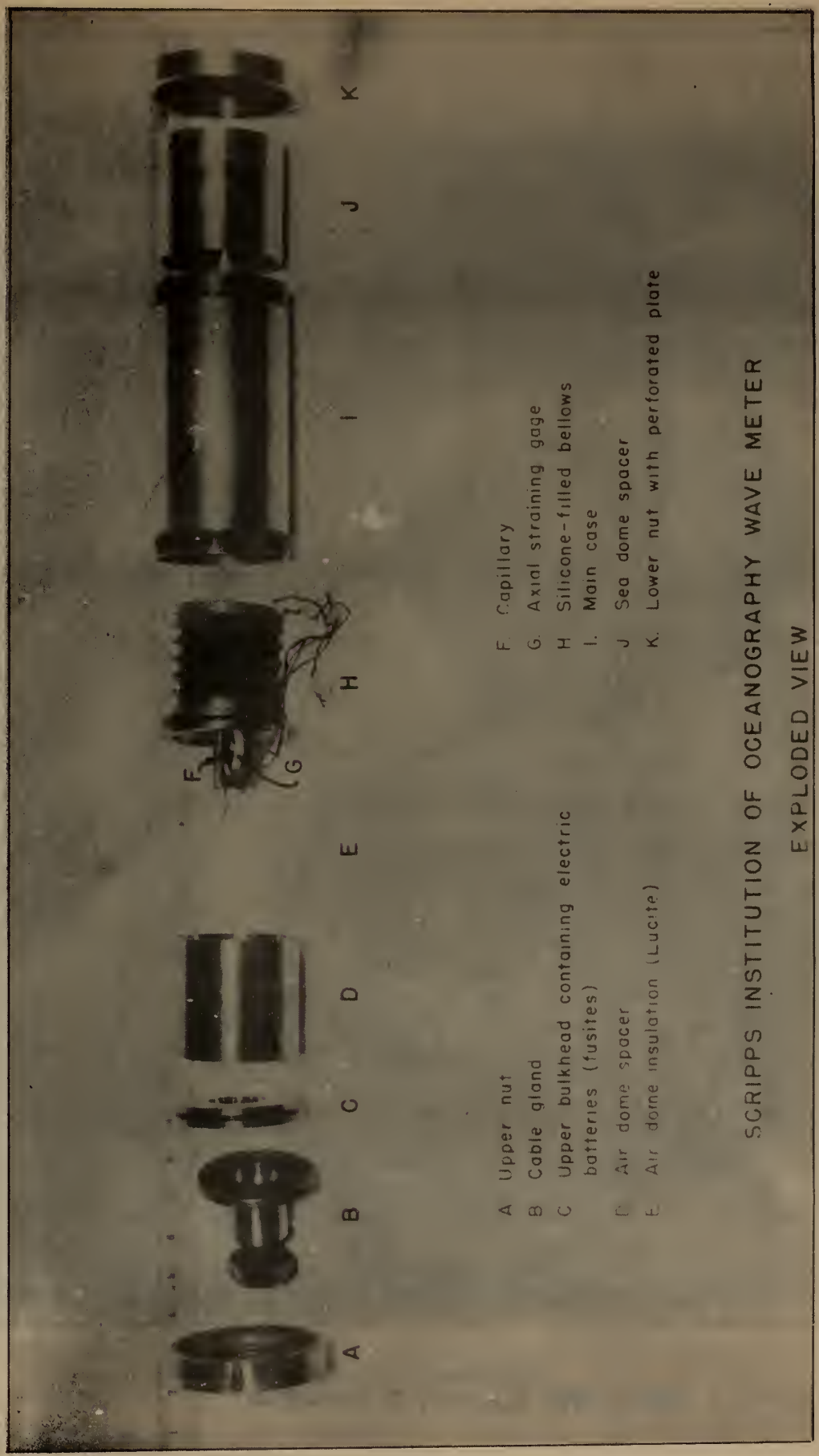

A
B
C
D
E
F
G
H
I
J
K
A Upper nut
B Cable gland
C Upper bulkhead containing electric batteries (fusites)
D Air dome spacer
E Air dome insulation (Lucite)
F Capillary
G Axial straining gage
H Silicone-filled bellows
I Main case
J Sea dome spacer
K Lower nut with perforated plate
SCRIPPS INSTITUTION OF OCEANOGRAPHY WAVE METER
EXPLODED VIEW

FIG. XIE-15

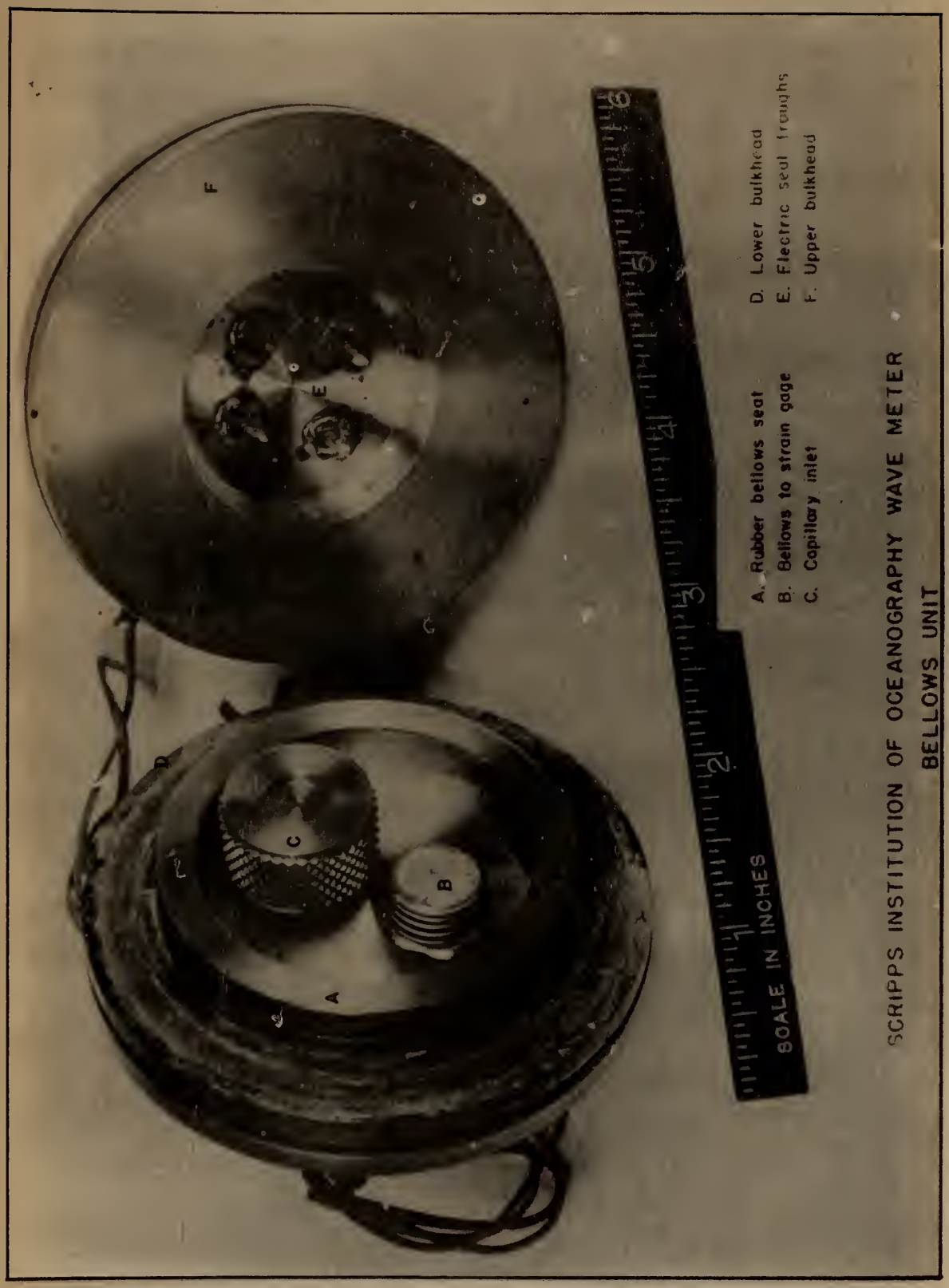
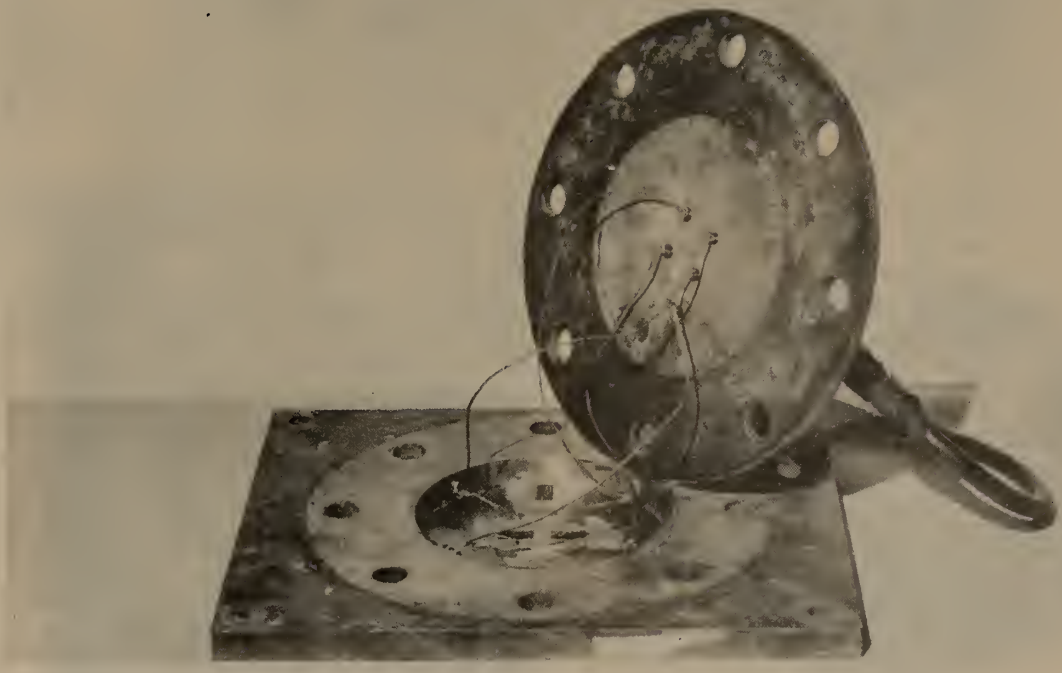

A. Rubber bellows seat
B. Bellows to strain gage
C. Capillary inlet
D. Lower bulkhead
E. Electric seal troughs
F. Upper bulkhead
SCALE IN INCHES
SCRIPPS INSTITUTION OF OCEANOGRAPHY WAVE METER
BELLOWS UNIT

FIG. XIE-16



MARK VI SHORE WAVE RECORDER



MARK VI PRESSURE HEAD

FIGURE VIE-17

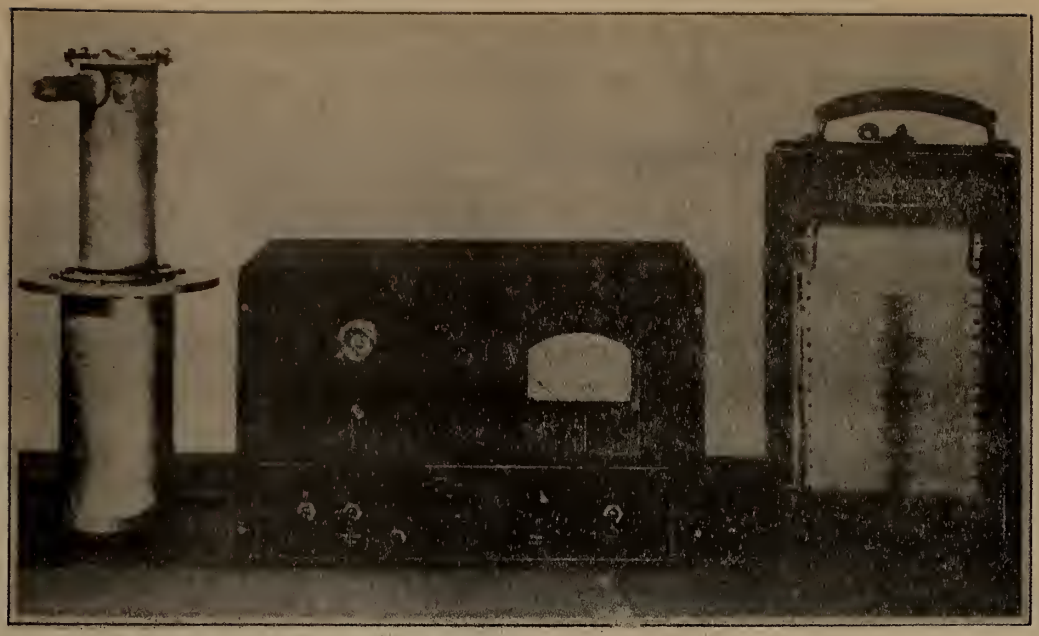
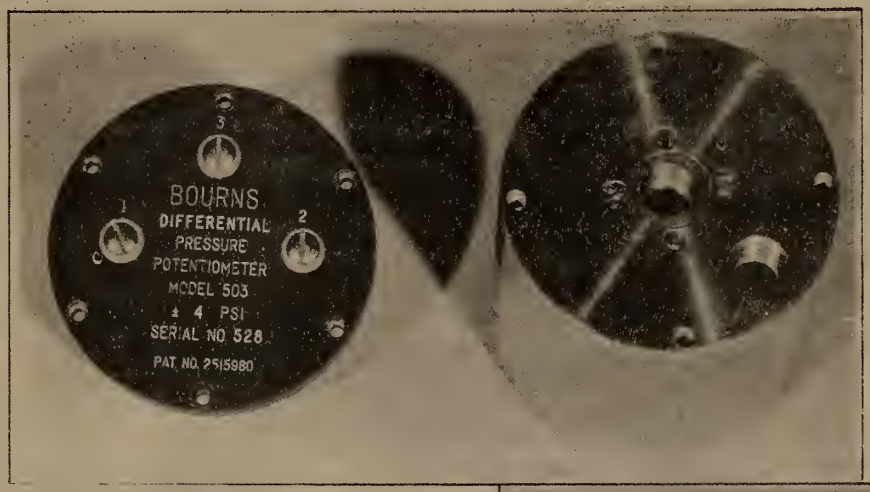


FIG. VIE-18 - Mark IX Shore Wave Recorder




a. Top and bottom view



b. Potentiometer and top



c. Internal bellows and casing

FIG. VIE-19 - Bourns differential pressure manometer

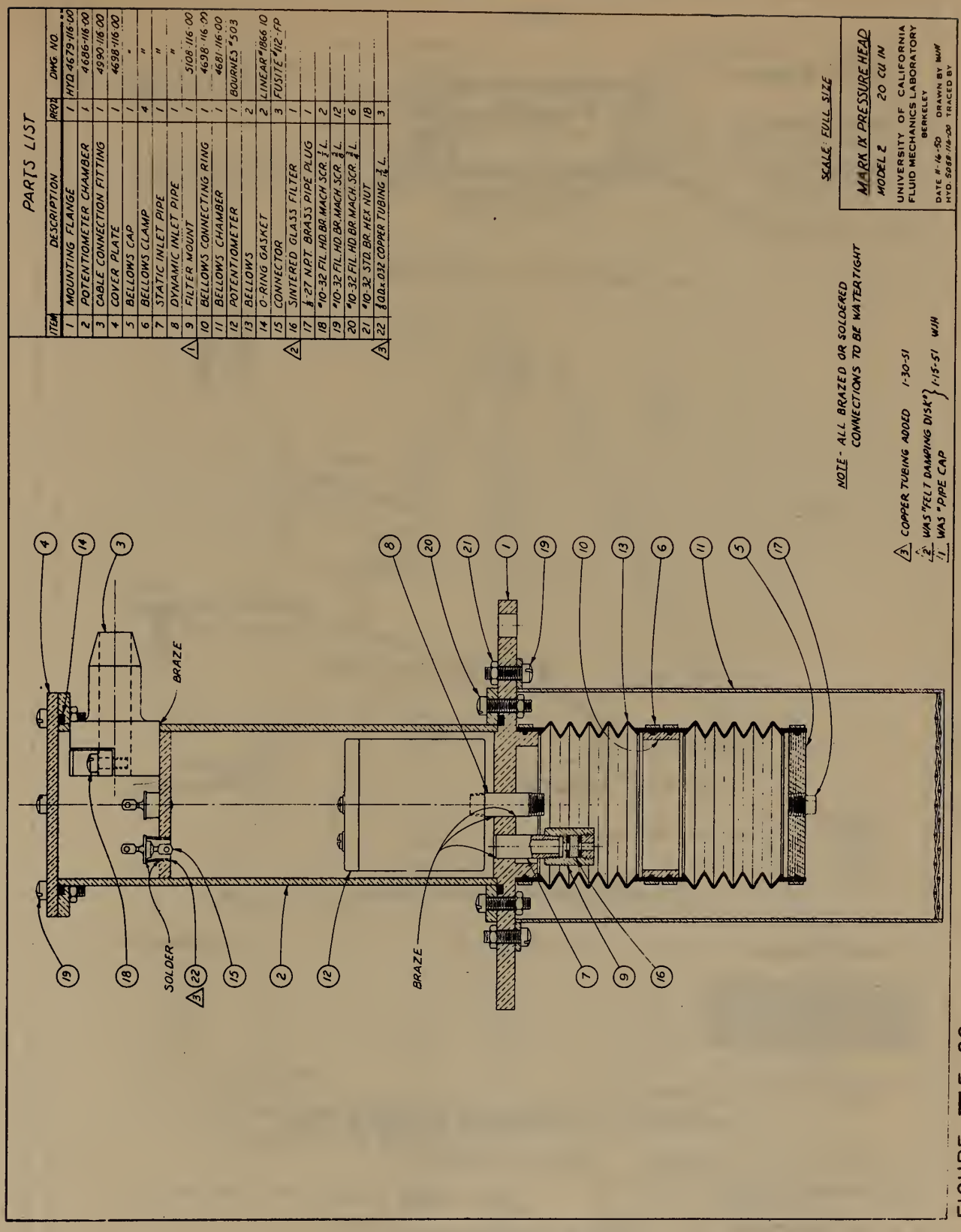
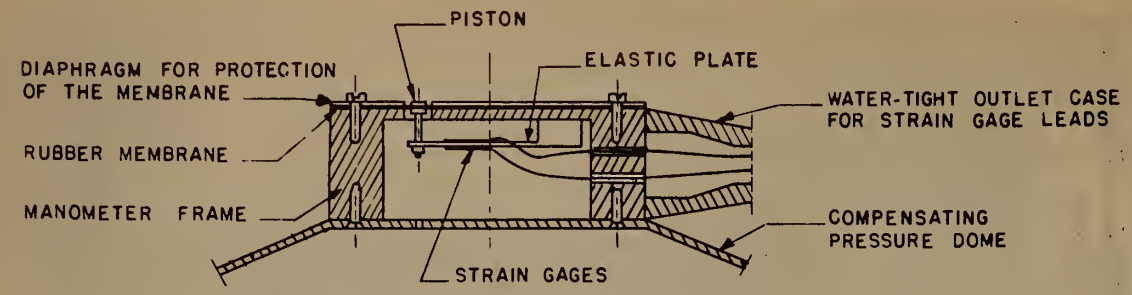
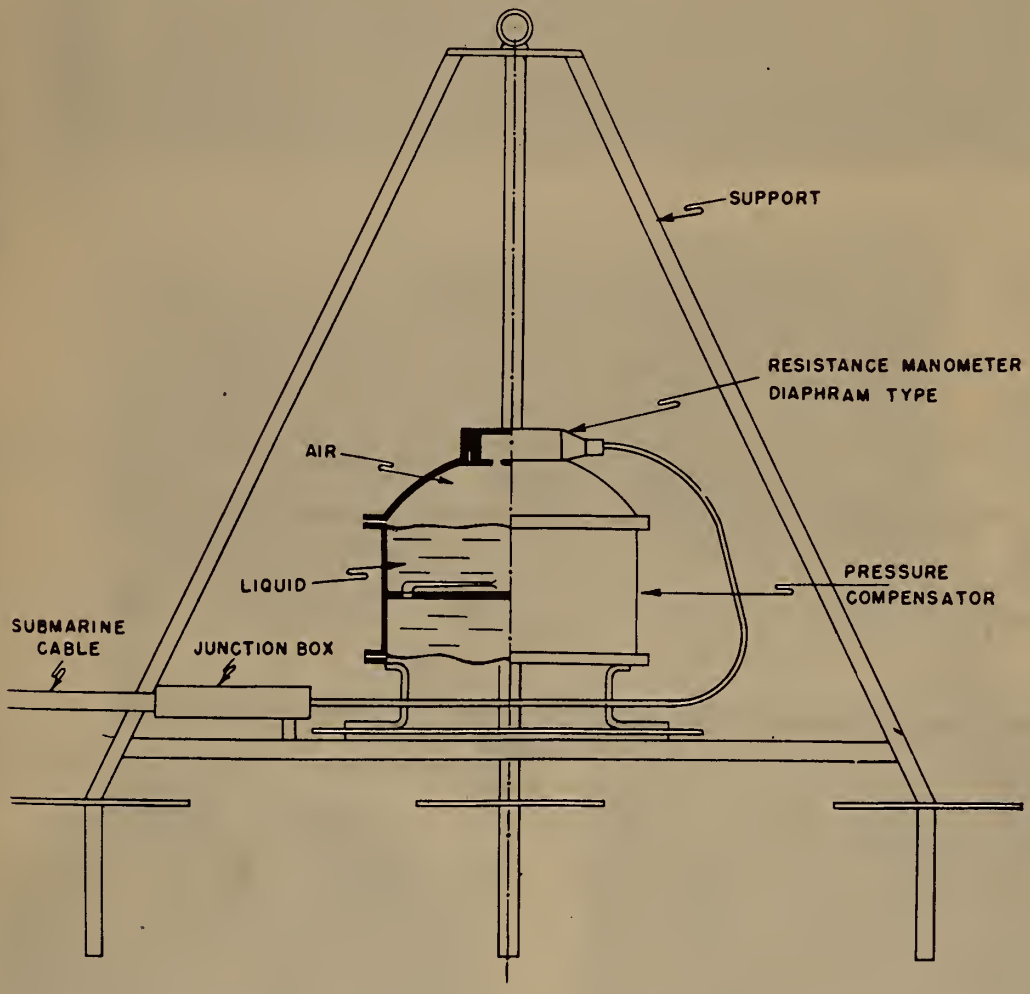


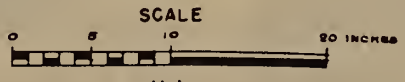

FIGURE VII E-20




(a) MANOMETER DIFFERENTIAL




NATIONAL HYDRAULIC LABORATORY
WAVE GAUGE




(b)

FIGURE VIE-21

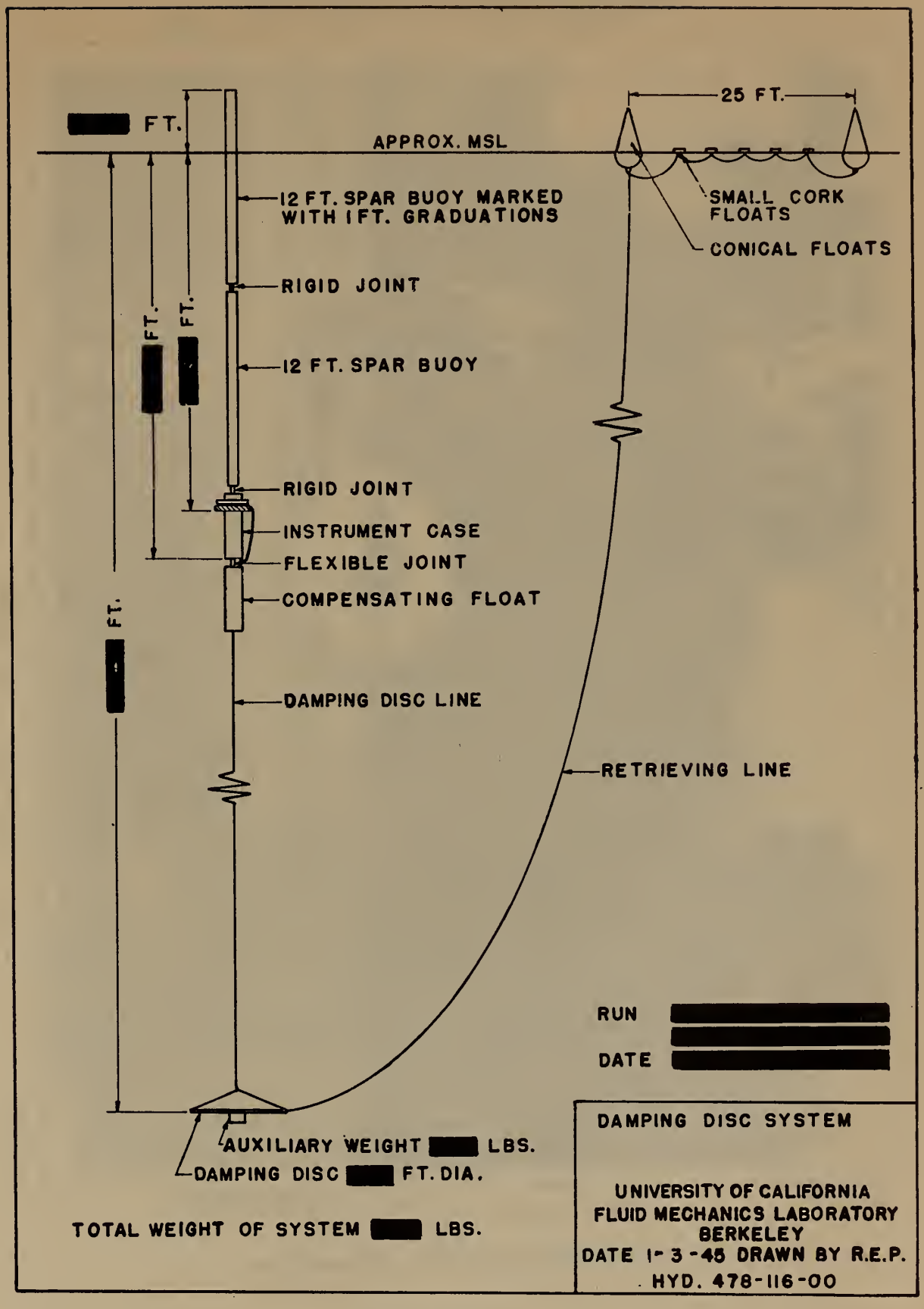



FIGURE VII E-22

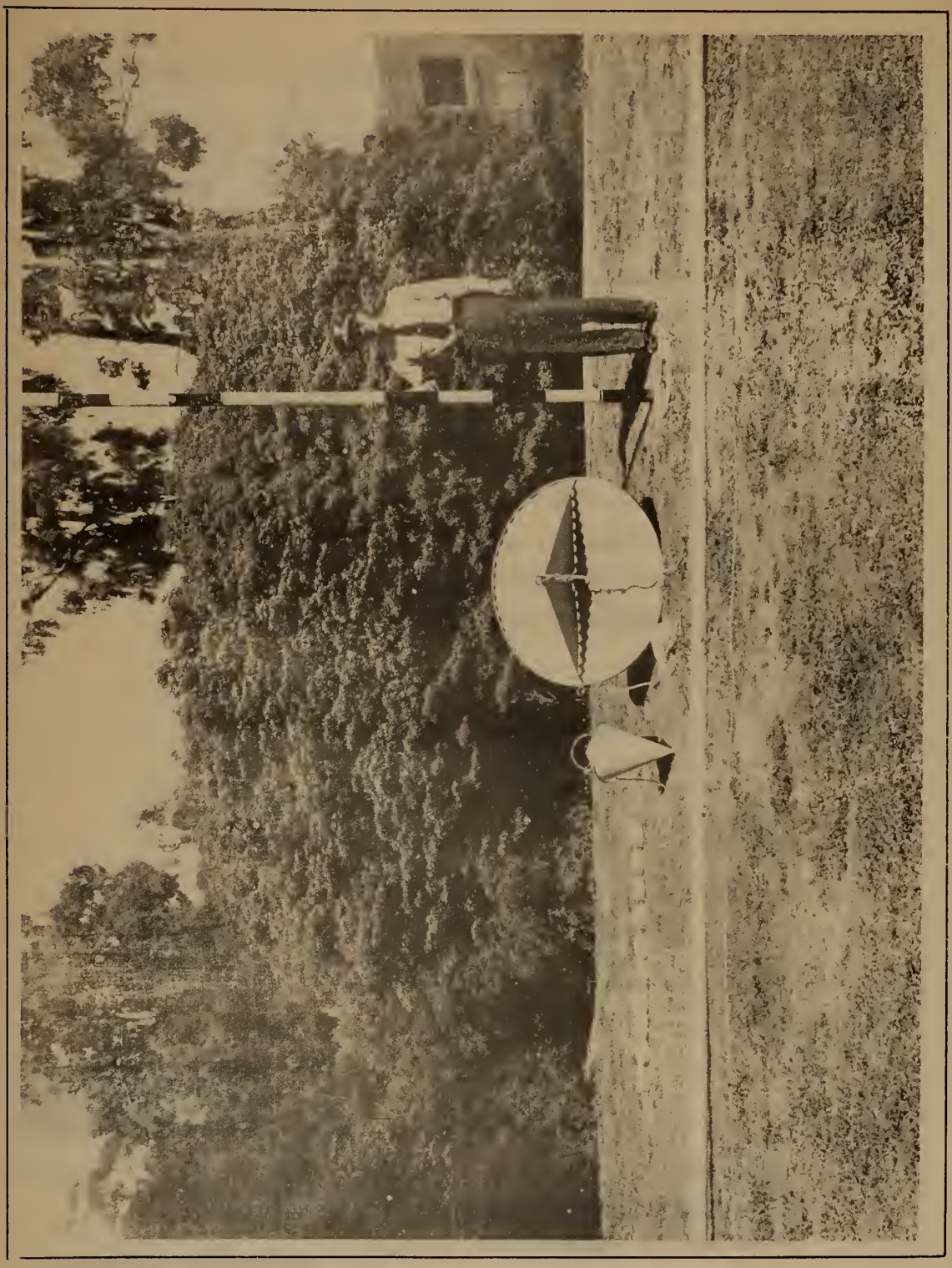


FIG. XII E-23 - Spar-buoy and damping disk system

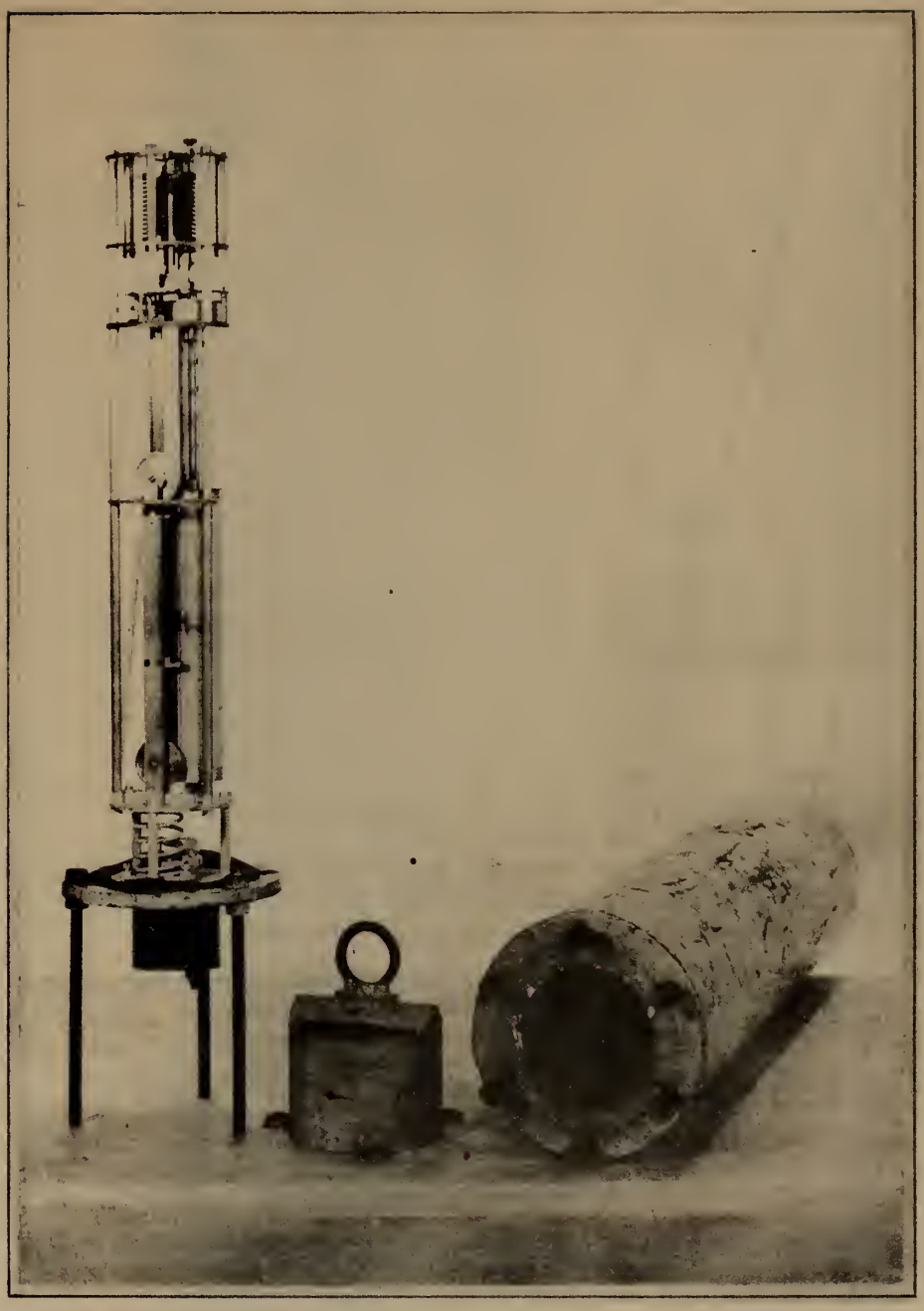


FIG. VIE-24 - Deep water wave recorder Mark I; improved model with clock drive

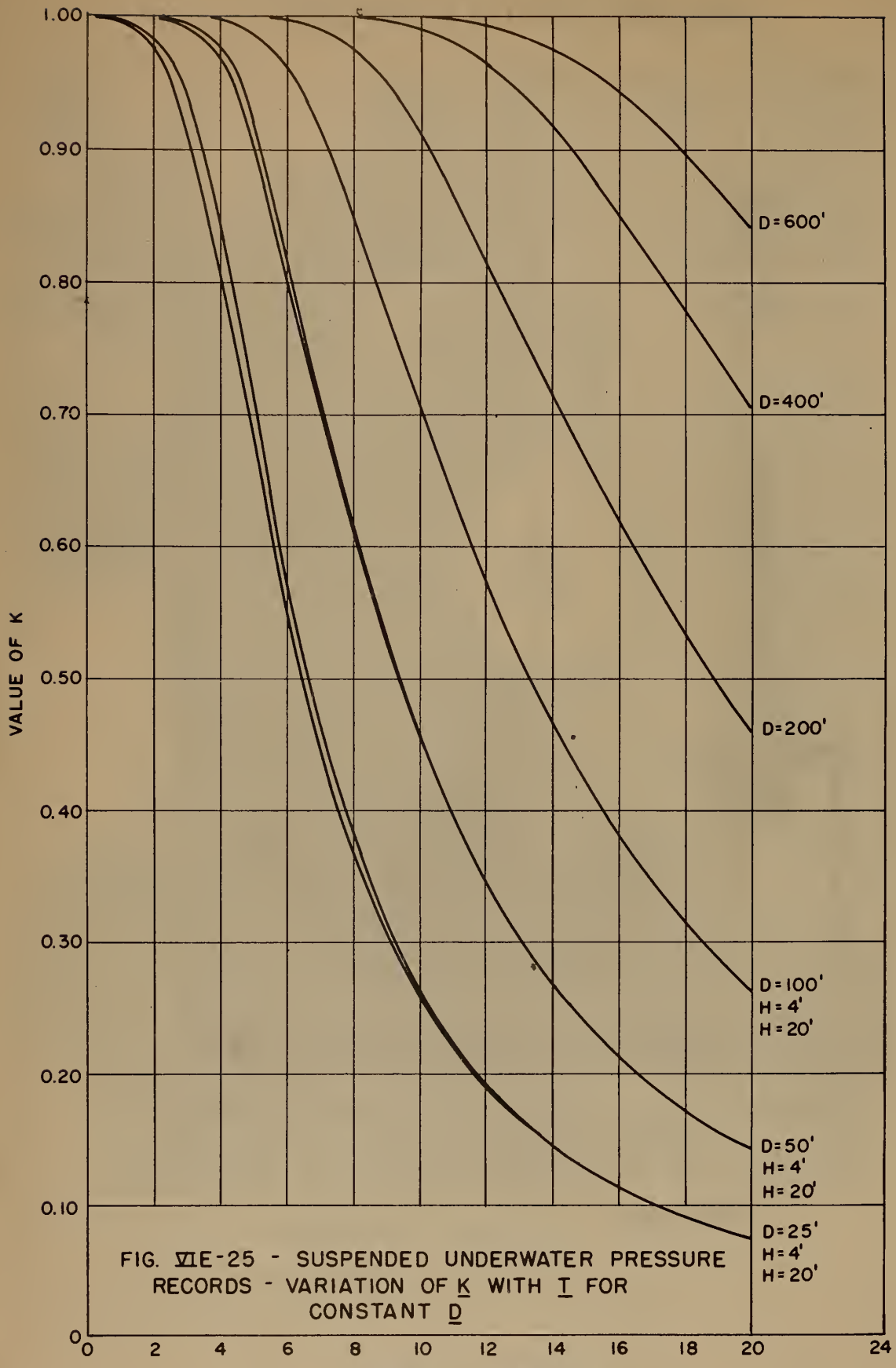



FIG. VIE-25 - SUSPENDED UNDERWATER PRESSURE RECORDS - VARIATION OF K WITH T FOR CONSTANT D



U.S. NAVY RADIO & SOUND LABORATORY
SAN DIEGO 52, CALIFORNIA

FIG. XII E-26 - Spar buoy

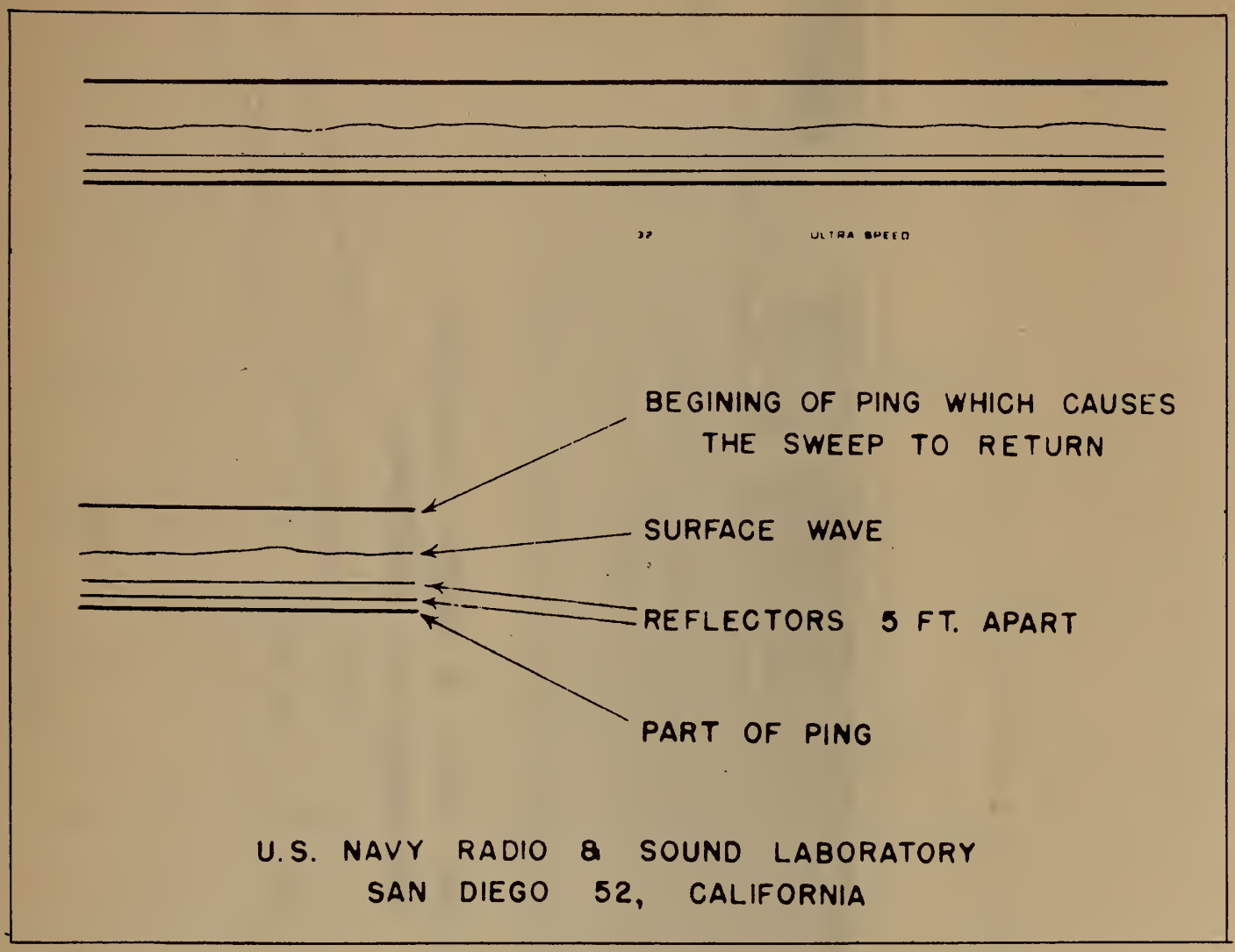



FIG. VIE-27 - Sonic record, sea test

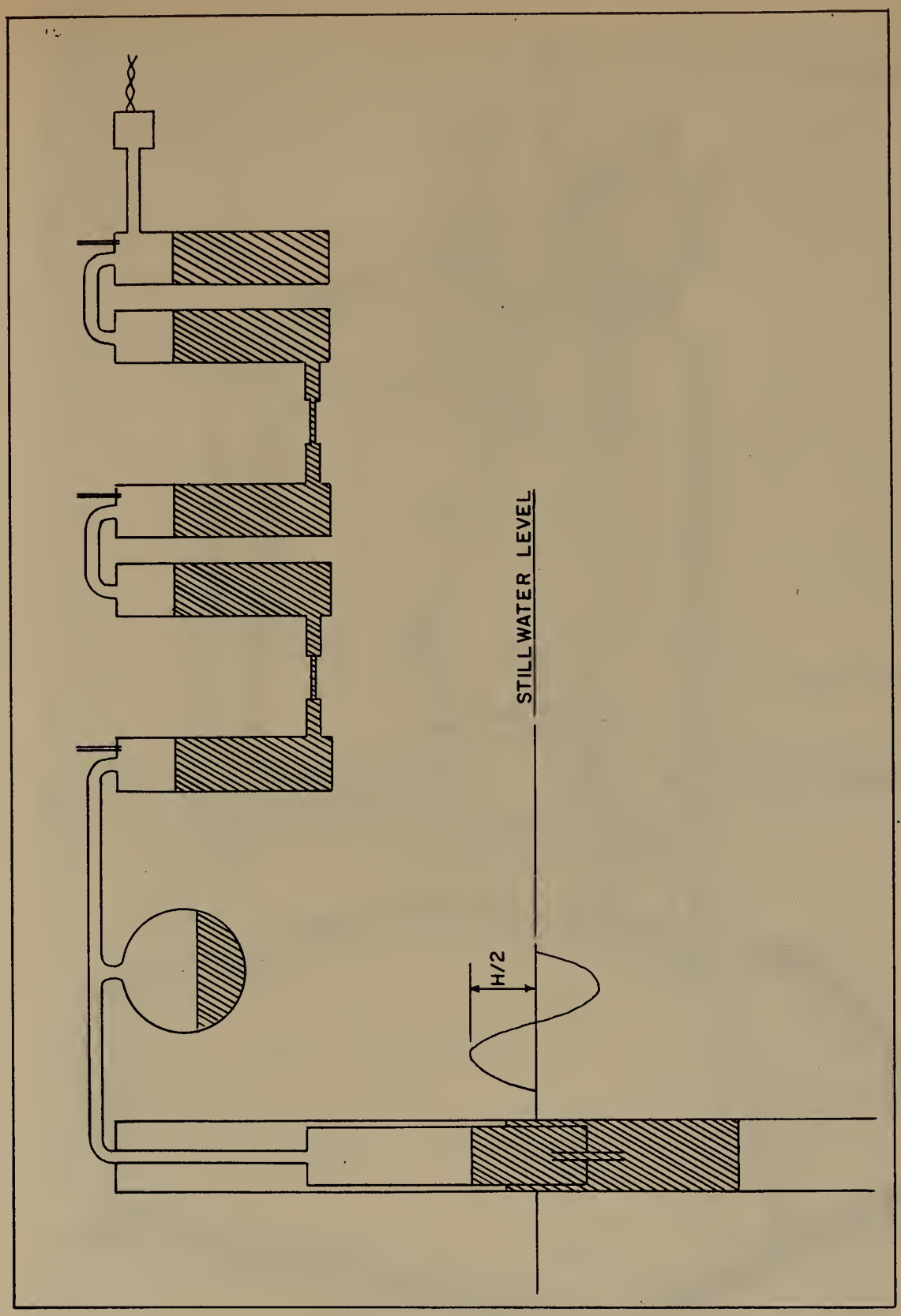



FIG. VII E-28 - Mark II Tsunami Recorder (schematic)

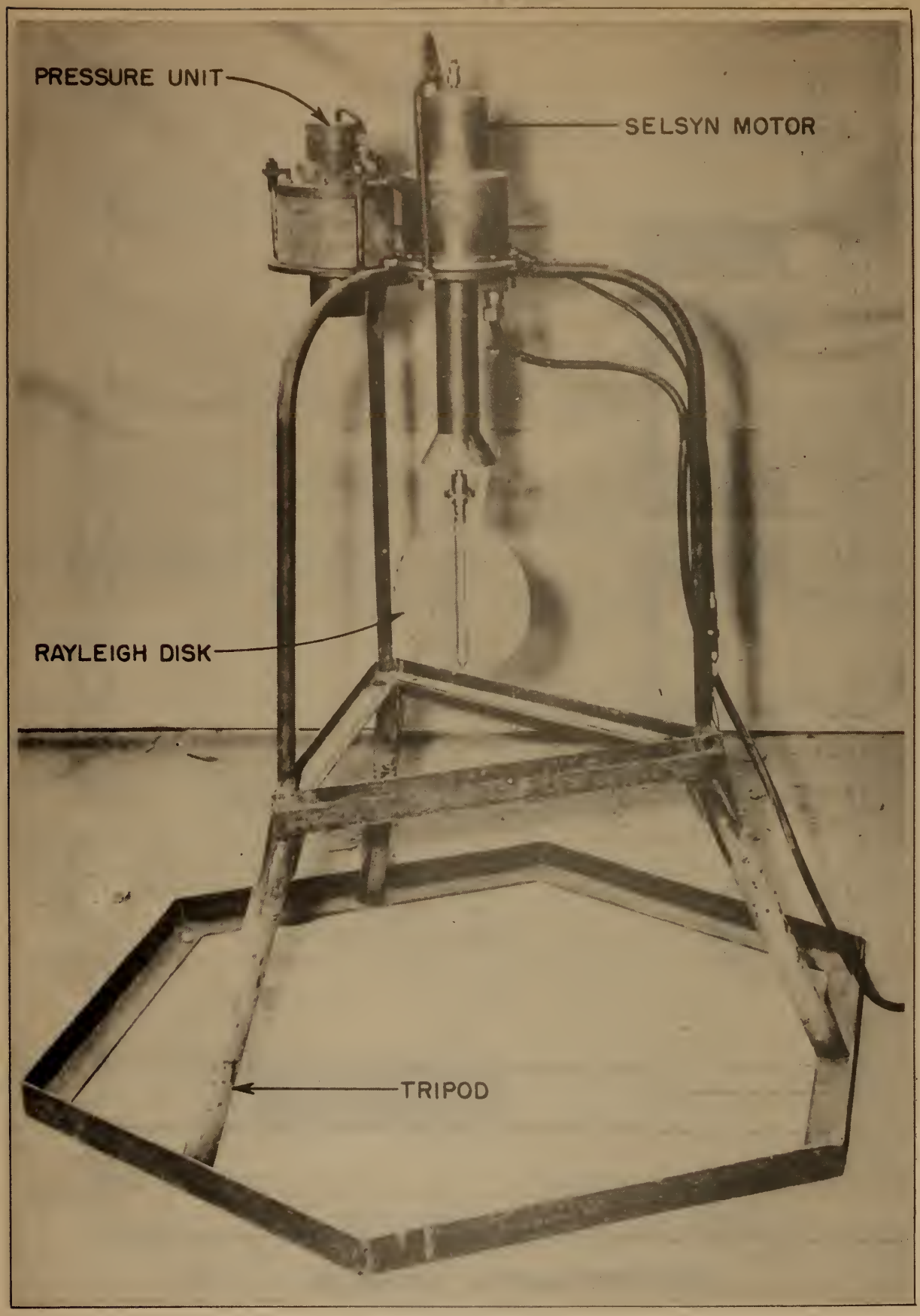



FIG. VII E-29

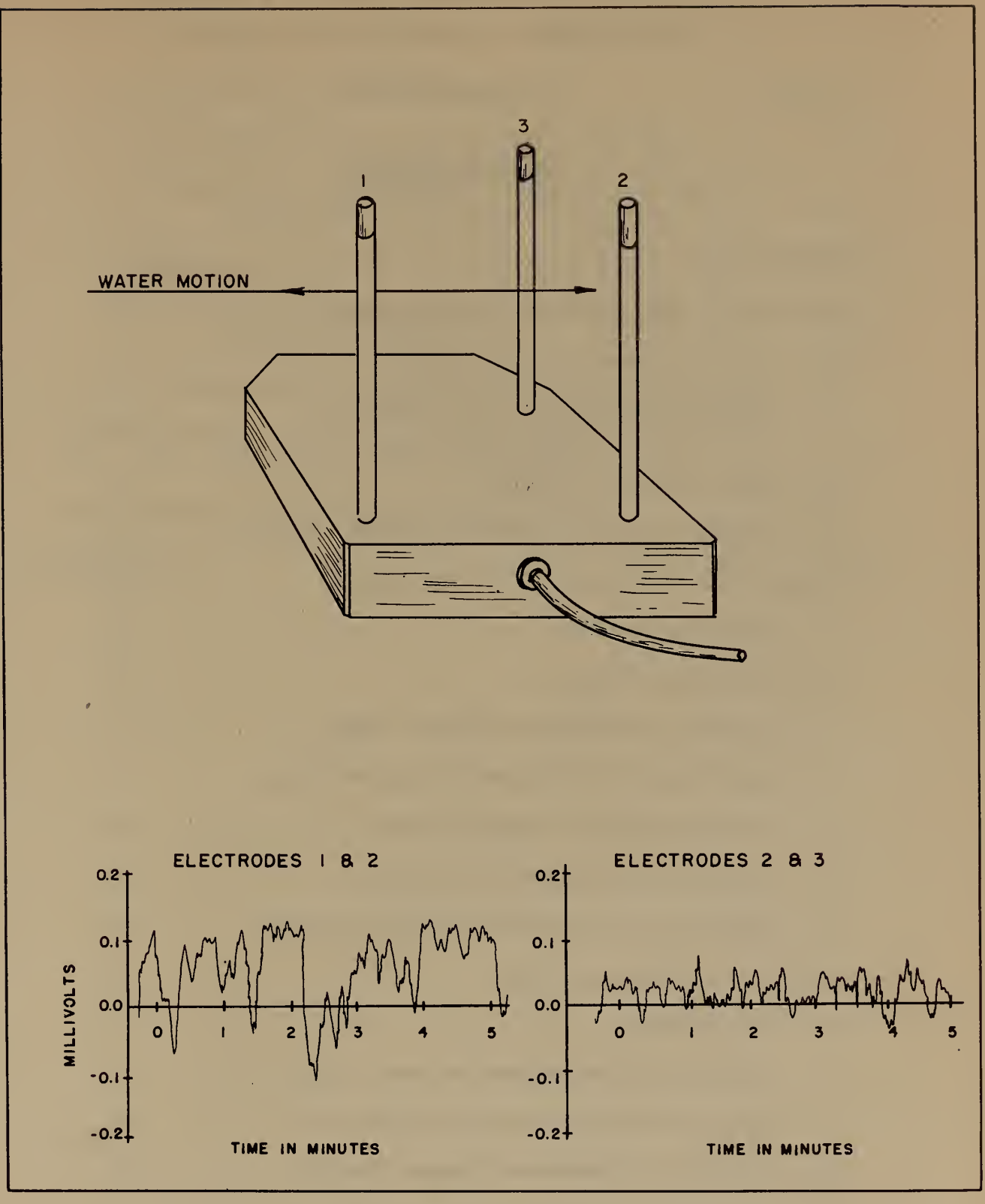



FIGURE VIE-30

MANUAL OF AMPHIBIOUS OCEANOGRAPHY

SECTION VI: INTELLIGENCE FOR AMPHIBIOUS OPERATIONS

# F. WAVE RECORDERS: INSTALLATION AND OPERATION

by F. E. Snodgrass

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## F. WAVE RECORDERS: INSTALLATION AND OPERATION

### 1. Introduction:

Installations of wave recorders for military applications can be classified under the two general headings of "temporary installations" and "permanent installations". Temporary installations of wave recorders would probably be made in order that wave height and period could be measured prior to and during amphibious landings. The wave recorder would be in use for short periods of time, seldom more than a few days. Permanent installation of wave recorders would probably be made to study waves acting at various military installations or to determine "wave climates" in various regions of interest to Intelligence. Permanent installations also have been made at amphibious training centers where repeated landings are made, and where military personnel are trained to forecast wave conditions from weather data.

### 2. Temporary Installations for Wave Recorders:

#### a. Need For Temporary Installation:

A reliable method of obtaining information on surf conditions during amphibious operations would be of great value. Serious accidents have occurred not only under combat conditions but during training operations which could have been averted by accurate determination of surf conditions. As even the trained observer can be deceived by breaker height, visual estimations are not reliable. In many cases, visual observation has resulted in underestimation and contributed to an accident.

Wave forecasting techniques developed over the past few years help supply data used in determining the probability of a craft traversing the surf zone without accident and, in cases of large landing forces, the probable percent casualty which will result from surf action. These forecasts should be checked by use of wave recorders located off the target beaches. The forecasts can be used to predict the following:

(i) average breaker height
(ii) maximum breaker height
(iii) average wave period
(iv) statistical variability of wave height and period

Utilizing weather data, the above information can be forecast with reasonable accuracy one or two days in advance. Perhaps some leeway can be allowed in the time of a scheduled landing if high surf is predicted or recorded. However, the landings must start just about as scheduled. If wave records could be obtained some distance seaward of the breaker zone and used for determining breaker heights a few minutes in advance, the schedule of each wave of landing craft could be varied slightly so that the surf crossings could coincide with periods of low breakers. Such last minute measurement of surf characteristics would considerably reduce the surf hazard.

There is an evident need for shore wave recording apparatus which can be simply and rapidly installed under combat conditions. With the recording unit operating aboard a control ship, such a device, used to direct the movement of amphibious craft through the surf zone during the period of lowest breakers, could greatly reduce casualities and would contribute to the success of landing operations.

b. Temporary Installations For Amphibious Operations:

On March 22, 1950 a wave recorder was tested at Fort Ord, California under simulated battle conditions (Reference VI F-5) (See Figure VI F-1).

"The UDT boat (LCPR) moved in directly toward the beach; just outside the breaker zone the craft came hard about and the pressure-head-sled was heaved overboard. The craft headed back out to sea at about 8 knots laying light 2 conductor demolition cable as it went. After about 1500 feet of cable was laid, the craft hove-to. The Speedomax recorder was hooked up to the cable and a wave record was being made within 10 minutes of the start of the shoreward run. The recording unit was supplied with power for the craft's wet-cell starting battery, converted to 110 volt a.c. by a vibrating converter. In making this test installation, the crudest sorts of devices were used; cable spool used pipe for an axle (held by two men), Speedomax merely sat on engine cover, etc. Total value of pressure head, cable, Speedomax recorder and converter is not over $1500. The simplicity of such a system is apparent."

"War-time or maneuver variations of this plan might be as follows:

(i) Pressure head could be installed and the cable laid by UDT as they leave the beach after their regular reconnaissance. (Explosion shocks probably would not hurt the pressure head.)

(ii) Craft could zig-zag if necessary while laying cable.

(iii) Several miles of cable could be laid from the small boat.

(iv) Cable could be buoyed at sea so that if recording craft had to cut and run it could be picked up again."

"If this technique becomes a permanent part of the amphibious operation, suitable cable reels would be installed in the craft assigned and a complete unit consisting of recorder and power supply would be part of the regular equipment. Perhaps the recorder would be mounted on a ship at the line of departure, perhaps on a control craft in close to shore. Many variations are possible."

The equipment used in the above tests also has been installed on several occasions as a temporary recorder with the recording unit on shore (Reference VI F-17) (See Figure VIF-2). The pressure head was carried by DUKW from the beach to a point just beyond the surf zone and dropped overboard. A 2 conductor demolition cable connected to the pressure head also was dragged off the beach by the DUKW. This cable served not only to electrically connect the pressure head to the recorder but also served as a retrieving line for the pressure head.

Patrick and Wiegel concluded their report with the following words:

> "All tests indicated that this assembly adequately meets the requirements of portability, strength and permanence of record. Unfortunately, the record is quite complex as the instrument picks up and records small surface fluctuations of three seconds and up with from 50% to 90% amplitude, depending upon the period.
>
> "As long as the recorder is a little beyond the surf zone it appears to respond fully to the waves travelling past. However, in the breaker zone, where the front of a wave is almost vertical, the pressure variation occurs during such a short time that the thermopile does not respond fully to the temperature variation which occurs in the bellows.
>
> "Upon examination of the pressure response curves in Report HE-116-108 of the Waves Project (Department of Engineering, University of California) it appears that, in surf less than 12 feet high, the instrument would be satisfactory if installed in twenty feet of water. At this depth, the shorter small waves would be hydrodynamically damped to a much greater extent than the waves of primary interest. As an example, in ten feet of water the pressure variation at the bottom is about 75% of the wave height for a five-second wave and 94% for a ten-second wave, while in twenty feet of water the response at the bottom is 55% for a five-second wave and 89% for a ten-second wave. Thus, the record of the ten-second and longer waves are almost the same in twenty feet of water as in ten, while records of the five-second period waves would be only 73% as great. The shorter periods would be even less.
>
> "It appears, therefore, that the Portable Mark V Wave Recorder should be used in water deep enough to damp out the short period waves, if present, and also out far enough so that the swell has not peaked up so that it gives a rapid pressure rise at the instrument with its corresponding rapid temperature rise in the bellows."

An installation with the wave recorder located on the beach as cited in the second example above has several disadvantages compared to an installation with the recorder located on the command ship directing the amphibious landings as cited in the first example. These disadvantages are as follows:

(i) the beach must be secured before a beach party can operate the equipment;

(ii) the wave data must be relayed from the beach party to the control ships directing the landings;

(iii) the pressure head cable stretched across the beach is apt to be damaged by the landing craft and vehicles on the beach.

c. Radio Link Wave Recorders:

The Scripps Institution of Oceanography and the Institute of Engineering Research of the University of California are both developing radio links to be used with their respective wave recording systems. The radio link will eliminate the necessity of laying a cable between the pressure head and the control ship. These systems will consist of the following component parts:

(i) a pressure head mounted in a small metal framework that will rest on bottom to measure the subsurface pressure variations;

(ii) an electrical cable that will connect the pressure head to a spar buoy floating on the surface;

(iii) a battery operated radio transmitter mounted in the spare buoy that will transmit a signal whose frequency is proportional to the subsurface pressure measured by the pressure head;

(iv) a radio receiver with a special recording attachment to record the transmitted signal on a graph calibrated in terms of the subsurface pressure.

The advantage of eliminating the underwater cable by the use of a radio link may be offset by the disadvantages inherent in any radio system. The most serious would probably be that an enemy could jam the transmission and prevent the reception of the required information. The fragility of the radio buoy would also be an important consideration. Whether or not the buoy would be damaged easily by enemy artillery fire would be a major concern. Certainly the buoy would be seen by the enemy and an attempt would be made to eliminate it either by radio jamming or by artillery fire.

d. Methods of Visual Estimating Breaker Height:

If wave recorders are not available during a landing operation, it may be necessary to visually estimate the height of waves in surf zone. Several visual aids have been developed to reduce the errors usually present in these observations. The beach pole method and the sighting bar method of measuring breaker height are discussed in Section VI-E (Wave Recorders) of this Manual.

Visual estimation of wave height is difficult since objects of known dimensions, to which the waves can be compared, are normally not found in the surf zone. Wave heights can be estimated with reasonable accuracy if this reference is supplied. Striped poles installed in the surf zone provide a suitable reference; considerable effort, therefore, has been made to determine the best method of installing these poles. Two practical methods (see Figure VI F-3) were developed (Reference VIF-18). The first method provides a technique of installing the poles by jetting them into bottom with a stream of water. The second method utilizes a sled which mounts a 20 foot vertical pole. The sled can be dragged to the desired location in the surf zone by a DUKW and retrieved by a cable attached between the sled and a shore anchor. Both of these methods of installing surf poles are discussed in detail in the above report.

3. Permanent Installation of Wave Recorders:

a. Selecting the Site:

The general location of a wave gage depends upon the purpose for which it is intended. If data are needed to compile statistical information describing the general wave action along a section of coastline, the gage should be located so that it is well exposed to the open sea. There should be no islands, bars, or prominent points to interfere with the waves before they reach the instrument. In this case, the most suitable sites are along straight beaches and at exposed points. If data are needed for a particular beach at which an amphibious landing is to take place, the recorder should be located near the beach.

The wave data obtained from gages exposed to the open sea can be used to estimate the amplitude of waves acting at a particular site along the same section of the coast if the wave direction is also known. Unfortunately wave direction must be determined by visual observation, or determined indirectly by the study of weather maps, since no satisfactory instrument has been devised to measure and record this factor. Visual observation is often made difficult by local wind chop which hides the more important swell coming from distant storms. Weather maps are employed to determine the location of the storms which generated the waves, thereby determining the direction from

which the waves must have approached the shore. The calculation of wave height at a particular site can then be made providing the following information is known:

(i) the wave height, period, and direction in deep water

(ii) the contours of the bottom over which the waves must pass to reach the site

These calculations are based on the principles of refraction and diffraction of ocean waves and are discussed in Sections I-D and I-E of this Manual. The effect of off-shore islands, irregular coastline contours, and interfering points usually can be determined with reasonable accuracy. These calculations normally are not made to compile data describing the wave action at a particular site but are made to determine possible critical wave conditions that cause convergence of the wave energy at a particular site.

Gages installed to study the waves acting at a harbor entrance, at a pier, or along a breakwater should be located near the site. Careful attention should be given to the local refraction to determine the relation between the waves at the gage and the site being studied. Often within a few hundred feet a noticeable difference in wave action can be observed. Gages not exposed to the open sea are seldom used in estimating the wave conditions offshore.

b. Pier Mounted Gages:

Piers provide supporting structures from which gages can be installed easily. Unfortunately piers are seldom built in locations exposed to the sea; rather, they are located in protected regions such as coves, bays, or where sheltered by offshore islands. At the same time many piers can be found that provide suitable locations for gages to study local wave action. For example, wave recorders have been located off piers at El Segundo and Huntington Beach, California, and the data obtained from them used to determine the wave action along the coast of southern California. When piers are available a surface type gage such as the Beach Erosion Board Step Resistance Gage can be installed which records the actual variation of water surface elevation (Reference VIF-6). Surface fluctuations caused by tides and local wind chop are recorded, and in many cases, may be desirable in the record. Pressure type wave recorders (Figure VIF-4) also can be installed easily off piers. By suspending the pressure recorder close to the water surface, pressure records corresponding closely to the surface profile can be obtained. Pressure type gages that either do or do not record tides can be used. The extent to which the pressure recorder will reproduce the locally-generated short-period waves can be controlled by adjusting the depth at which the gage is installed below the surface.

In some locations off-shore oil well structures have been built that provide mountings for wave gages. Also, an installation has been completed at Cape Henry, Virginia (Reference VIF-27) in which a step-resistance gage was attached to a piling specially driven for the purpose of installing the gage. The piling, 60 foot in length,

was driven 25 feet into the sand bottom at a site 2500 feet from shore; the water depth was 20 feet. The gage was connected by armored cable to recorders located at a shore station.

Surface type gages, although requiring cleaning at three to four month intervals, can be operated for several years without major repair. Several pressure type gages are also available that provide continuous service for at least one year without repair or maintenance. Either type gage should be installed at least three piling diameters from the nearest pile.

c. Tripods For Subsurface Pressure Gages:

Pressure type gages, supported by small tripods resting on the ocean bottom, provide a practical method of recording the wave action where there are no piers. The tripods vary in design depending upon the material available, the equipment available for handling the tripod during installation, the size and type of marker buoy to be used, and the size and shape of the pressure head. The two primary requirements of the tripod are as follows:

(i) The tripod should have sufficient weight to keep it in place on bottom. Marker buoys, when acted upon by heavy seas, may exert a considerable upsetting force on the tripod, if the marker buoy cable is attached to the top of the tripod. Also, the electrical cable usually attached to the base of the tripod may tend to drag the tripod when the cable is acted upon by long shore currents. Normally tripods weigh between 250 pounds and 2000 pounds depending primarily on the marker buoy size and the equipment available for handling the tripod.

(ii) The tripod should have sufficient height to prevent the "sanding down" of the pressure head: sand movements are known to occur considerable distances offshore, often changing the bottom elevations by several feet. This is especially true of the seasonal movement of sand onshore during the summer months and offshore during the winter months.

The Woods Hole tripod (Figure VIF-5) (Reference VIF-32) has a special design feature which allows the instrument to be detached from its concrete base in case the unit becomes covered with sand and cannot be lifted without possible breaking of the lifting line. The tripod is made in two parts: a concrete base weighing approximately 300 pounds and a pipe framework which supports the pressure head. A shear pin whose strength is less than that of the lifting cable but of sufficient strength to lift the concrete base, holds the two halves of the structure together. A lifting line of 3/8" wire rope is attached to the top of the pipe framework and to a marker buoy. If an attempt is made to lift the tripod while it is covered with sand the shear pin will fail and only the pipe framework need be lifted to recover the instrument.

The concrete base of the Woods Hole tripod is provided with a cavity directly below the tripod in which cable can be coiled. A sufficient amount of cable is stored in this cavity to reach the surface of the water, allowing the instrument to be removed and a replacement connected without pulling off the bottom any cable which might also be covered with sand.

Figure VIF-5 also illustrates two typical tripods used by the Waves Research Group of the University of California. These tripods are usually five to seven feet in height with the instrument located four to five feet above the base. Scrap metal or cast concrete blocks are used to increase the tripod weight. The lifting cable is attached between the top of the tripod and a marker buoy.

d. Marker Buoys For Subsurface Pressure Gages:

In the past the methods of attaching the lifting line and marker buoy have been very unsatisfactory. Usually the tripod was lowered to the bottom by a 3/8" to 1/2" wire rope and a buoy attached to the lifting line after the tripod was in place on the bottom. To provide working cable to pass over the hoisting frame and attach to the winch, the lifting cable length was normally made twice the depth of the water. Several difficulties arose from this type of installation. The continual working of the cable due to the buoy following the surface waves weakened the cable and eventually caused failure. To reduce this action a small buoy with buoyancy just sufficient to remain afloat under the weight of the cable was used. These small buoys would still break the cable under the action of large waves, especially after several months of exposure of the cable to the salt water.

Marker buoys installed according to Coast Guard specifications using chain between the anchor and the buoy, certainly would last a longer time but would require larger boats to make the installation and a larger tripod to serve as an anchor. Typical Coast Guard specifications for a small open-sea type buoy would be as follows: 3rd class special nun buoy, 656 pounds; 3/4" chain; maximum water depth 14 fathoms; chain length 2½ times water depth; 2000 pound concrete block anchor. An installation of this type should be serviced once each six months (paint buoy and check chain). The chain should last between one and two years depending upon the amount of wave action.

Greatest wear of the chain occurs between the links that touch bottom during wave troughs at low tide and the links that are lifted off bottom during wave crests at high tide. This wear is due to the rotation of each link as it is lifted. Additional chain may be lifted off bottom during storms and high winds, but the percentage of time is small and the wear of these links is not as critical as that wear caused by normal wave action.

By using a chain whose entire length cannot be lifted by the buoy allows the use of a relatively small anchor. The anchor serves only to prevent the chain from being dragged along the bottom during large storms. A long chain also prevents any snapping action caused by the buoy lifting all the slack out of the line. This snapping action is probably the greatest cause of cable failure when small round buoys are used with wire cable lifting lines.

The inconveniences of handling larger anchors, buoys and cables, as normally used by the Coast Guard, have prevented their use. A small, light, marker-buoy system that is proving to be more satisfactory comprises a short line and a spar type buoy. The basic idea behind this scheme is to provide a buoy that holds the short line taut at all times to prevent continual working of the line and to prevent failure by snapping action. One such scheme used by the Beach Erosion Board employs a wooden spar buoy approximately five feet in length which is connected to a 3/8" wire cable (see Figure VIF-5). The length of the cable is adjusted according to the water depth so that the buoy is exposed only at low tide. A positive net buoyancy is assumed at all times except possibly during trough of large waves at low tides.

A second example of the short-line spar-buoy scheme is provided by an installation made at Point Pinos, California, by the University of California (Figure VIF-6). This system employs a 30 foot, metal spar-buoy six inches in diameter, and a five foot length of chain. The chain is connected between the top of a six foot 1500 pound tripod and the buoy. The length of the chain is adjusted so that the top of the buoy is exposed at low tides. A net positive buoyancy exists at all times and is equal to 150 pounds at the connection between the chain and the tripod when the buoy is completely submerged. This installation was checked after five months service and was found to be in good condition; very little wear of the chain had taken place.

The disadvantage of the short-line spar-buoy system is twofold:

(i) Special provisions must be made to lower and lift the instrument, and

(ii) The replacement or cleaning of the buoy requires lifting the instrument.

The first difficulty is easily overcome. The instrument can be lowered by a separate line attached to the tripod by a hook which will free itself when the instrument reaches bottom. The instrument can be lifted by lowering a chain noose around the buoy to contact the buoy chain near the top of the tripod. When the noose is pulled tight by the lifting line, the two chains inter-link so that the tripod can be raised.

The second disadvantage of the short-line spar-buoy system, the inconvenience of lifting the instrument for periodic cleaning of the buoy, cannot be overcome simply. Periodic cleaning must be made to prevent excessive sea growth on the buoy. If the growth is allowed to accumulate the downward drag force exerted by the motion of the water may be sufficient to overcome the net buoyancy with the result that snapping action could take place.

A third scheme combining the features of the short-line spar-buoy type marker and the Coast Guard buoy type marker has been developed by the University of California (see Figure VIF-7). This system utilizes (i) a medium size spar buoy (15 feet long, 7 inches

in diameter, 149 pounds total weight), (ii) a small chain (3/8 inch) connecting the buoy to a section of large chain (1 inch) which acts as a "variable anchor", (iii) a lifting chain (3/8 inch) between the one inch chain and the tripod, and (iv) a 1200 pound tripod (see Figure VIF-10). The buoy size is determined by the amount of chain to be supported which includes a section of the large chain. The system is designed so that at a -1 foot tide, the buoy is exposed two feet; at a +4 foot tide, the buoy is totally submerged. The large chain therefore, acts as a "variable anchor" in that it reduces the motion of the spar buoy, maintains tension in the buoy chain, and prevents snapping action in the chain.

By adjusting the length of the buoy chain to a length equal to the water depth (at MLLW) less 18 feet, the amount of one inch anchor chain lifted off the bottom will be sufficient to submerge the buoy at a +4 tide. This can be shown as follows (with reference to Figure VIF-10).

Buoyant force of the spar buoy (B) = $17.2\,\ell_1$ pounds

Weight of the anchor chain = $9.3\,\ell_2$ pounds

Weight of buoy = 150 pounds

Weight of buoy chain (in air) = 46 pounds

Therefore

$$17.2\,\ell_1 = 9.3\,\ell_2 + 190 \qquad \text{(VIF-3.1)}$$

and

$$\ell_1 + \ell_c + \ell_2 = (\text{MLLW}) + T \qquad \text{(VIF-3.2)}$$

If the buoy submerges, $\ell_1$ will be equal to the length of the buoy (15 feet) and the amount of anchor chain lifted will be:

$$\ell_2 = \frac{17.2\,(15) - 190}{9.3} = 7.3 \text{ feet} \qquad \text{(VIF-3.3)}$$

By adjusting the length of the buoy chain the buoy will be submerged at the desired tide stage of four feet. Equation VIF-3.2 now becomes:

$$\ell_c = \text{MLLW} + 4 - 15 - 7.3 = \text{MLLW} - 18.3$$
$$\ell_c \tilde{=} \text{MLLW} - 18 \qquad \text{(VIF-3.4)}$$

The length of anchor chain lifted off bottom for any water level can now be determined from equations VIF-3.2 and 3.4.

$$\ell_1 + \ell_2 = \text{MLLW} + T - (\text{MLLW} - 18)$$
$$= T + 18 \qquad \text{(VIF-3.5)}$$

and

$$\ell_2 = 0.649T + 4.14 \qquad \text{(VIF-3.6)}$$

Equation VIF-3.6 indicates that the water level must be about $6\frac{1}{2}$ feet below MLLW to cause all of the anchor chain to rest on bottom.

e. Specifications For Submarine Cable:

A cable that has been used along the Pacific Coast for several years and has proven satisfactory except under very unfavorable conditions has specifications as follows:

Simplex Wire and Cable Company

Anhydrex S A Neoprene Galvanized Armor; Jute Covered Submarine Cable

(i) Conductors #14-7 strand #22 tinned

(ii) 5/64" wall Anhydrex S A colored cotton braid cable - fill waterproof jute

(iii) Tape

(iv) 6/64" wall Neoprene

(v) Tape waterproof jute serve

(vi) #10 BWG galvanized armor

(vii) Waterproof jute serve

These cables have shown only slight wear over a period of one to two years. One cable which was continually exposed at the uprush of a cobble beach failed in approximately four months. The combination of the alternate exposure of the cable to the water and air, and the abrasion of the cobbles caused a rapid corroding of the armor and the subsequent cable failure.

The most suitable cable available on the commercial market for installations where the cable may be subjected to abnormal abrasion

and large tension forces is built by the Simplex Wire and Cable Co. The specifications of this cable are as follows:

Anhydrex SA Neoprene Galvanized Armor

Neoprene Jacketed Submarine Cable

(i) Conductors #14 - 7 strands #22 tinned

(ii) 5/64" wall Anhydrex SA colored cotton braid cable - fill.waterproof jute

(iii) Tape

(iv) 6/64" wall Neoprene

(v) Tape waterproof jute serve

(vi) #10 BWG galvanized armor

(vii) 5/64" wall Neoprene

(viii) two serves seine twine

(ix) 7/64" wall Neoprene

(x) approximate O.D. 1.66 inches; 2320 lbs/1000 ft.

f. Installation of Electrical Cables:

The installation of the electrical cable between the offshore tripod and the shore station is a relatively simple task under favorable conditions and with proper equipment. Favorable conditions include a straight sandy beach without reefs, and a sand bottom from the beach to the instrument. Under these conditions an armored cable can be laid along the bottom without anchors or additional protection and little or no cable wear will take place. The cable will quickly "sand-down" so that it is not exposed to the turbulent action of the surf zone or to currents that may exist at times parallel to the beach. After a few months the cables usually will be covered by sand to such depths that they cannot be recovered.

The "sanding down" takes place for two reasons; first, the fluidity of the sand (especially in the surf zone) will allow the cable to settle until it is covered to a depth of a few inches or more, and second, the movement of sand bars, the movement of sand onshore and offshore, and the movement of sand along the shore, will alternately under cut and bury the cable until it may be covered to a depth of several feet. During the summer when the sand moves toward the beach, the offshore cable may be very near the sand surface, or actually exposed. Directly under the summer berm on the beach the cable may be buried as much as ten feet. Conversely, during the winter the cable

may be near the sand surface in the surf zone while offshore it may be buried several feet. The cable seldom, if ever, will be exposed along its entire length.

The greatest danger to the cable exists during storms when the waves approach the shore at a sharp angle. The waves may cut a scarp several feet deep (see Figure VI F-8) which will expose the cable to the turbulence and littoral currents of the surf zone during the storm.

Under favorable conditions the armored cable need be used only to cross the surf zone. Unarmored cable can then be spliced to the armored cable and laid along the bottom to the instrument site. The splice normally does not have the full strength of the cable and care must be taken to prevent tension at this point. Anchoring the cable near the splice will reduce the tension in the splice. An installation of this type was made at Oceanside, California, (Reference VI F-31) which was described as follows:

> "A two thousand foot section of four-conductor armored cable was used through the surf zone with three thousand feet of two conductor demolition cable, spliced to the end of the armored cable, for the remainder of the distance. The armored cable was faked, in the shape of a figure 8, into the cargo compartment of a DUKW with the bottom ten feet left sticking up along side of the craft's gunwale. This was spliced to the top end of the demolition cable which had been faked into the cargo compartment of a second DUKW. The bottom end of the demolition cable then was spliced to the Mark V, No. 1 which, mounted in an iron stand, was placed in the second DUKW. All splices were completed while the craft were still on the beach. The remaining thousand feet of armored cable was laid out on the beach and a bight taken on it with a rope which was attached to the DUKW. This remaining cable was to be pulled out to sea by the DUKW as it went through the surf. In order that there would be no tension on the splice, the demolition cable had been doubled back for about three feet and securely bound to the armored cable.
>
> "The system of using a heavy armored cable through the surf zone with a light cable running the remainder of the distance is satisfactory for the type of conditions encountered at this location (flat, sandy bottom). It is cheaper because the relatively inexpensive demolition cable can be used for most of the distance. It is easier to install because little of the difficult-handling armored cable is used. It was found that the installation was easier to perform when all splices had been made ashore before starting."

When reefs exist along the beach (see Figure VIF-8) or the bottom offshore is covered with rock, the cable cannot sand-down and special anchoring of the cable is necessary to hold the cable in place. Anchoring prevents wear of the cable on sharp edges of rocks and prevents excessive tension in the cable due to long lengths of cable being exposed to underwater currents. The problem of rocky bottoms offshore can be dealt with effectively by having a deep sea diver walk the cable, laying it around and between the rocks and perhaps anchoring the cable periodically with concrete blocks set on top of the cable. No satisfactory technique has been developed for installing cables over reefs in the surf zone.

g. Installation of Electrical Cables by DUKWs:

The length of cable usually installed for a wave recorder varies between one thousand and five thousand feet depending upon how far offshore the desired depth of water can be found. Cable-laying ships cannot be used economically to install these short lengths of cable. It is, therefore, necessary to use small boats, landing craft and amphibious vehicles to perform this operation. The DUKW is used more often than the other craft since it can be loaded with the cable, driven to the shore site and used to lay the cable in one operation. The remarks which follow apply directly to the use of a DUKW to install the cable but should also apply in general to the use of any small craft.

Figure VIF-9 shows cable spool holders designed to mount 2500 foot spools of armored cable in the cargo compartment of a DUKW. The design includes a foot brake which acts on the rims of the spool to hold tension in the cable as it is being laid. Without the foot brake the spool will spin due to the weight of the cable hanging in the water and cable will be unreeled faster than the craft progresses.

The cable can also be faked in the DUKW as illustrated in Figure VIF-10. By faking the cable (coiling the cable to form a "figure eight") as shown, the cable can be played out without twisting. A full spool of armored cable (2500 feet) can be stowed in the DUKW cargo compartment without difficulty.

The disadvantages of taking cable in the cargo compartment are as follows:

(i) There is always a possibility of the cable becoming tangled and a danger to anyone attempting to free such tangles.

(ii) There is no simple way to hold tension on the cable being laid and the cable tends to uncoil too rapidly.

(iii) Considerable work is involved in unspooling and recoiling the heavy armored cable.

Attempts have been made to drag a portion of the cable offshore from a spool mounted on the beach. Not more than 1000 to 1500 feet of armored cable can be dragged by a floating DUKW unless floats are attached to the cable to reduce the bottom friction. The following quotations taken from the field notes of a DUKW party illustrate some of these techniques and the difficulties that can be encountered.

Bascom (Reference VIF-3) (See Figure VIF-11): Installation of a wave recorder at Point Arguello, California.

> "In accordance with the plan, the DUKWs entered the water at the Arguello surf station, moved north the four miles of water to Point Arguello, tied up side-by-side and anchored in the lee of a large rock. Radio contact was established between the five men in the DUKWs and the three on the cliff. The DUKWs launched a skiff with one man aboard. The end of a pilot line ($\frac{1}{2}$ inch manila) was lowered into the skiff by means of a light linen line slung between two rocky promontories. The skiff took the pilot line to the DUKWs. With considerable effort, and help in turning the cable spool from the cliff crew, the end of the submarine cable was hauled aboard the DUKW. About 1200 feet of the cable was then flemished down between the two DUKWs in the following manner: the cable was too heavy to pull off the cliff by manpower; therefore, the DUKWs would tow off about 300 to 400 feet of cable, the crew would pull it aboard and flemish it down. This would, of course, pull the DUKWs slowly backwards towards the cliff. When they attained their original position they would take a new bend on the cable and tow out a few hundred feet more to repeat the process. When the required 1200 feet was aboard, the DUKWs took a firm grip on the cable; a 55 gal. drum was attached to the cable on the cliff. Thenceforth, the DUKWs pulled the cable straight south from the cliff, stopping at 250 foot intervals for the crew on the cliff to attach another drum. After the last of the 10 drums was attached, the DUKWs continued to pull off cable (unbuoyed), swinging about to the west at the same time. When another 800 feet was off, the drum was braked and the unsupported cable allowed to settle to the bottom. The DUKWs then headed for the buoys which marked the cable course; the floating line of 2500 feet of cable gradually swung around to follow this course. While tension was kept on the cable, the float-barrels were cut free. The grip on the cable was then released and the 1200 feet of cable on the DUKWs was allowed to play out over the stern as they moved seaward on the cable course. The end of the cable was sealed and buoyed with a large round marked buoy and cast off. The cable served as its own anchor and the buoy was picked up in the same position the next day."

Bascom (Reference VIF-2) (See Figure VIF-12): Installation of wave recorder at Heceta Head, Oregon:

> "Sea was calm ($H_B$ = 5', T = 10") and the day was clear; there was a light westerly wind. The green DUKW with the spool of cable aboard was backed out to the water's edge. The pressure head was fixed in the triangle and the final connection made

and tested. The #511 submarine cable was fastened to the triangle by means of special brackets; the cable was to be towed by a cable sock which was fastened to the cable about ten feet from the triangle. The sock was connected to the grey DUKW with ten feet of 1/4" steel cable. The grey DUKW hoisted the triangle in its "A" frame and put out through the surf. The first heave carried the cable about 1,000 ft. offshore. At that point the sock slipped on the cable and put the strain on the triangle. To avoid any damage to the instrument, the submarine cable was cut and the DUKW returned to the beach. The cable was retracted and another splice made. The grey DUKW then carried a bight of cable to about 800 ft. offshore, dropped it and returned for the triangle. The procedure was the same as before but the sock was securely fastened this time. This time, the DUKW got to about 1500 feet offshore (30 ft. of water). By this time a rather strong wind had come up that carried the DUKW south of its course; the triangle was lowered and buoyed and the party secured."

Bascom (Reference VIF-4): Installation of wave recorder at Quillayute, Washington:

"The cable was moved out onto the beach and set up in the spool frame so that it would rotate freely. About 1800 feet of the seaward end was flemished down in the hold of a DUKW, and secured there. The DUKW then put out to sea towing the cable behind it. It was hoped that enough cable would be unreeled and dragged off the beach to complete the cable laying in a single operation. Such was not the case. After about 1500 feet was off, the DUKW could move no more; the line securing the cable to the DUKW was cast off and the 1800 feet aboard the DUKW was laid out along the course. The empty DUKW came back ashore, seized a bight of the cable and towed about 1000 feet more out to sea. This was a mistake. Apparently underwater obstructions or rapid sand changes held it fast, for although the outer end of the cable was picked up with a grappling hook, the DUKW was unable to tow any more cable out to sea. After landing, it was found to be impossible to winch any part of the cable back ashore. This means that about 1000 feet of the cable is still in a large loop in the surf zone, probably lost forever. Enough cable was then unlaid from the reel to adequately complete the shore end. The remaining cable was flemished down on the DUKW, and taken out to sea. The seaward end of the laid cable was recovered and a splice was made; the rest of the cable was then laid off the DUKW and buoyed."

## 4. Analysis of Pressure Records*

### a. Introduction

Due to the irregularity of ocean waves in height and period, the definition of terms employed in describing the waves is very important. The following basic definitions (Reference VIF-12) have been accepted:

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*This entire section has been taken from Reference VIF-26

(i) Wave height is the vertical distance between the crest and the preceding trough.

(ii) Significant wave height is the average height of 33 1/3 percent of the highest waves.

(iii) Wave period is the time interval between the appearance at a fixed point of successive wave crests.

(iv) Significant wave period is the average period for the well-defined series of highest waves observed.

(v) Wave direction is the orientation of the line of travel of the largest well-defined waves.

The definitions of significant wave height and period are incomplete in that the length of observation is not specified. A minimum period of ten minutes has been suggested by Scripps Institution of Oceanography (Reference VIF-21) while a minimum of twenty minutes has been suggested by R. C. Folsom (Reference VIF-12). Little evidence has been published as to the effect of the length of the observation period but, as in measuring any statistical quantity, the longer the period taken the better the results, providing conditions do not change during the period. However, a long period of observation means greater expense in regard to the amount of chart paper used in recording data and the amount of time required for analyzing the records.

b. Analysis of Wave Records For Wave Height

In analyzing the wave records made at various locations along the Pacific Coast during the last several years, the following step-by-step procedure has been used by the University of California, Berkeley:

(i) Select a continuous record of about a twenty-minute duration which was made while the recorder was operating at fast speed (three inches per minute) and at approximately the time for which wave information is desired.

(ii) Determine the total number of waves recorded during the selected interval by:

(a) Measuring the wave period of all well-defined waves in the selected interval of the record and computing an average value.

(b) Computing the total number of waves by dividing the determined average wave period into the total seconds covered by the record interval.

(iii) Determine the significant wave height by:

(a) Measuring the height of the highest 33 1/3 percent of the total number of waves in the selected interval.

(b) Computing the average height of the measured values.

(iv) Determine the average height of the highest 10 percent of the waves as in step number three. (Optional)

(v) Determine the maximum wave height in the selected twenty-minute period.

Several studies have been made to determine the statistical distribution of wave height and periods about the significant wave heights and period. These studies do show a tendency for the wave heights to follow definite statistical distributions. Thus, the measurement of a significant height can be used to describe the wave heights of the irregular wave trains.

Wave period distribution studies have not been as successful. The distribution of wave periods about an average, or significant period, is not consistent from one record sample to the next. In this sense significant wave periods, as determined from the above, do not describe the wave periods of the irregular waves found in nature.

One of the first of these studies, made by R. L. Wiegel (Reference VIF-30), showed that a definite relationship existed between the average height of the highest one-third of the waves, the average height of the highest one-tenth of the waves, and the highest wave that occurred in the 20 minute interval.

The ratio of the average height of the highest 10 percent to the average height of the highest 33 1/3 percent has been found to be 1.29. This ratio was determined from wave data recorded at Point Arguello, California (1.30), Point Sur, California (1.27), and Heceta Head, Oregon (1.30). A substantial number of the daily values of this ratio agreed within 10 percent with the average value derived from readings taken over a fourteen-month period at Point Sur and Heceta Head, and over a three-month period at Point Arguello.

Also from the above data, the average ratio of the maximum wave height to the average wave height of the highest 10 percent was found to be 1.46; while the average ratio of the maximum height to the average height of the highest 33 1/3 percent was found to be 1.87. Again a substantial number of the daily values of these ratios compared favorably, agreeing within 20 percent with the average value.

In a tabulation of wave data taken from two widely separated stations in the Atlantic (Cuttyhunk, Massachusetts and Bermuda) by H. R. Siewell (References VIF-25 and 22), a constant of 1.57 was found for the average ratio of the average height of the highest 30 percent to the average height of all waves.

The agreement of daily values to the average values of these ratios and the agreement among values determined at widely separated stations indicated that a definite statistical grouping of waves is generated by wind.

Evidence to further substantiate this theory is found in the results of a statistical analysis conducted by R. R. Putz (Reference VI F-19) at the University of California, Berkeley. Analysis was made of twenty-five wave records selected from various localities and made at various times of the year to obtain good sampling. Putz found that the statistical frequency distribution of observed wave height in a twenty-minute interval is approximately constant in form and, for a first approximation, requires for its complete description only the determination of a typical height, such as the "significant wave-height". The wave-height distribution of all twenty-five pressure records matched, with reasonable accuracy, a Pearson Type III frequency function with a 0.8 positive skewness and proportionality of the mean and the standard deviations.

Utilizing this mathematical model, Putz computed values for ratios reported by R. L. Wiegel and H. R. Seiwell. The value of maximum wave height determined from the model was taken as the probable maximum wave in two twenty-minute intervals as used by Wiegel in determining his daily maximum wave height. Excellent agreement was found among these three sources as shown in the following table.

TABLE VI C-1

Comparison of Wave Height Ratios for Various Pressure Recorders and a Frequency Function

| Basis of Calculations | Computed Ratios | | | | Remarks |
|---|---|---|---|---|---|
| | $\frac{H_{1/3}}{H_{ave}}$ | $\frac{H_{1/10}}{H_{1/3}}$ | $\frac{H_{max}}{H_{1/3}}$ | $\frac{H_{max}}{H_{1/10}}$ | |
| Point Arguello, Calif. wave recorder | | 1.30 | 1.85 | 1.42 | 3 months of data |
| Point Sur, California wave recorder | | 1.27 | 1.85 | 1.46 | 14 months of data |
| Heceta Head, Oregon wave recorder | | 1.30 | 1.91 | 1.47 | 14 months of data |
| Cuttyhunk, Massachusetts wave recorder | 1.57 | | | | 10 months of data |
| Bermuda wave recorder | 1.57 | | | | 4 months of data |
| Average of wave record values | 1.57 | 1.29 | 1.87 | 1.46 | |
| Pearson Type III frequency function MODEL | 1.57 | 1.29 | 1.81 | 1.41 | Model based upon 25 selected records |

c. Analysis of Wave Records For Wave Period:

Analysis of wave records for the characteristic period is accomplished by measuring the average period of the larger, well-defined waves appearing on the record. This is comparable to measuring the significant height of the waves by determining the average height of the highest one-third of the waves. The characteristic period of the waves does not describe the period-distribution, however, as the characteristic height describes wave-height-distribution. Although wave heights have been found to follow a simple mathematical distribution even though the waves may be arriving from two or more storm areas, wave periods do not follow a simple distribution if more than one generating area exists. Additional information is needed to adequately describe wave periods.

The need for more accurate methods of analyzing wave periods has led to the development of electrical-mechanical analyzers, (i) a frequency analyzer, (ii) an auto-correlation function analyzer.

The frequency analyzer measures the presence of the various sinusoidal frequency components in the record to produce a frequency distribution curve. Even though this analysis may give an accurate mathematical representation of the data, the validity of its physical representation has been questioned by H. R. Seiwell (References VI F-23 and 24). A study of the frequency distribution curves of pressure type wave recorders by the Oceanographic Research Group, Admiralty Research Laboratory, Teddington, England (Reference VI F-1), and later by W. H. Munk (References VI F-15 and 16) indicates that this type analysis is useful in tracking storms and in correlating meteorological and wave data.

The second type of analyzer which is based on the auto-correlation function, has been investigated by the Marine Physical Laboratory, University of California (Reference VI F-20), San Diego, California, and the Woods Hole Oceanographic Institution (Reference VI F-14), Woods Hole, Massachusetts. Although still in the process of development, this method shows promise of more accurately describing the physical characteristics of surface waves than the frequency analyzer.

d. Analysis of Underwater Pressure Records:

The analysis of pressure records for wave period is the same as the analysis of surface wave records. The records differ, however, in that the short period waves are not registered to the same degree as the long period waves by pressure recorders due to the hydrodynamic pressure attenuation of the water. As a result, many of the shorter period waves may not appear on the pressure record.

If the technique of measuring the periods of only the larger, well-defined waves of the record is followed (as described under Analysis of Wave Records for Wave Period), the measured period will be approximately the same as would be obtained if the record were made with a surface type gage. For locations on the exposed coast, the short period waves, not recorded by pressure, are generated by local wind.

In several cases, attempts have been made to utilize the hydrodynamic attenuation of short period waves by installing gages in deep water (about 600 feet) so that only the waves of long periods (storm forerunners, seiches and tsunamis) will be recorded. These long period waves are recorded by pressure heads installed in shallow water, but are "lost" in the record of shorter period waves. Installations of this type of instrument have been made, but due to instrument difficulties no satisfactory records have been obtained.

To obtain the surface wave heights from the pressure record, two factors are required: (i) the calibration of the instrument and (ii) the pressure response factor relating the subsurface pressure fluctuations to the surface wave. Thus, if

$H$ = wave height at the surface (in feet);

$C_i$ = calibration factor of the instrument (expressed in feet of water pressure variation per chart division);

$K$ = pressure response factor based on the depth of the instrument, the depth of the water and the length (or period) of the wave being recorded;

$R_i$ = reading of the instrument;

the following equation is used to obtain the surface wave heights:

$$H = C_i/K\,(R_i) \quad \text{. . . . . (VI F-4.1)}$$

The calibration factor for most instruments in use today is a constant which is independent of wave period and depth of the instrument. The instrument provides a record of the pressure variations at the instrument which is accurate in amplitude and wave form.

The relation of the subsurface pressure fluctuations to the surface wave has been determined theoretically for two dimensional, irrotational motion of an incompressible fluid in the relatively deep channel of constant depth (Reference VI F-11). The response factor K has been shown to be:

$$K = \frac{\cosh 2\pi d/L\,(1 - z/d)}{\cosh 2\pi d/L} \quad \text{. . . . . (VI F-4.2a)}$$

where

$z$ = depth at which the pressure variation is being measured (in feet),

$d$ = depth of water at the instrument (in feet),

$L$ = length of the surface wave (in feet).

When $z = d$, the pressure variation is measured at the bottom and equation VI F-4.2a reduces to:

$$K = \frac{1}{\cosh 2\pi d/L} \quad \ldots\ldots\ldots\ldots\ldots\ldots \text{(VI F-4.26)}$$

Pressure records do not enable the direct measurement of wave length; the wave length must be calculated from the wave period using the following equation:

$$L = (gT^2/2\pi) \tanh 2\pi d/L \quad \ldots\ldots\ldots\ldots\ldots \text{(VI F-4.3)}$$

where T = wave period (in seconds).

Suitable graphs and tables (Reference VI F-28) are available (see Sections IIB and C of this Manual) for the solution of these equations. Graphs have been prepared which enable the response factor (K) to be determined if the water depth (d), instrument depth (z), and wave period (T) are known.

Two errors arise when the above equations are used to determine the response factor (K) for ocean waves: (i) an average or significant period must be used in the equation while the actual wave period is continuously varying and individual waves are not sinusoidal in form; (ii) wave heights computed from these equations have been shown by several observers to be from six to twenty-five percent too low.

Considering the first of these two errors, greater accuracy probably could be attained if the pressure response factor were determined for each wave and the equivalent surface wave were individually computed. This procedure might be feasible from a practical standpoint if the statistical distribution of wave height and wave period could be established so that fewer waves need be analyzed to completely describe the state of the waves.

The second of these two errors emphasizes the need to reconsider the basic theory which does not agree with experiment. Every observer who has simultaneously measured the surface waves and the subsurface pressure fluctuations has found the theoretical response factor determined from equation (VI F-4.2a) to be too small. Ten random measurements made at the Waterways Experiment Station indicated an average correction of 1.07 should be applied to the response factor. Seventeen laboratory measurements at Berkeley indicated an average correction of 1.10 (Reference VI F-12). Field data reported by the Woods Hole Oceanographic Institute (References VI F-22 and 23) indicated a correction factor in excess of 1.20 while the three sets of field data obtained by the University of California (Reference VI F-10) indicated values of 1.06, 1.08, and 1.18.

e. Example of Statistical Analysis of Records:

The wave recorder is programmed to run at slow-speed (3 inches per hour) for 5 hours and 35 minutes, and at fast-speed (3 inches per minute) for 25 minutes. When definite and pronounced increases in wave amplitudes (indicating the arrival of wave trains) are evident on the slow-speed portion of the record, the time and date of these arrivals are noted by the analyzer. Only the fast-speed sections of the chart are analyzed for wave height and period. A twenty minute interval is selected for determining characteristic wave period and maximum and

characteristic wave heights. Except in cases of storm arrivals, as previously mentioned, the records are analyzed at twelve-hour intervals. Records are analyzed every six hours during storm periods or when the slow speed portion of the record indicates rapidly changing conditions.

The following steps in the procedure for analyzing wave records have been developed over a period of several years at the University of California:

(1) Receipt of the Record:

The graphic chart represents the time history of the surface wave action and should be the final authority in case of future conjecture as to the validity of statistical information compiled therefrom. Hence, a system of logging charts is used for facility of future reference. The log of records contains (i) the time and date the run began and ended, (ii) the number of the chart (i.e., its chronological sequence) and (iii) the date the record was received. Following is a form which has been used for logging records.

| LOCATION | | START | | END | | REMARKS |
|---|---|---|---|---|---|---|
| Received | Roll No. | Time | Date | Time | Date | |
| | | | | | | |

Log for Mark IX Wave Recorder Charts

(2) Reading the Chart:

(a) Frequency of Taking Samples:

(i) The wave records are analyzed during the fast-speed portion of the chart at approximately twelve-hour intervals. Recorders are programmed to obtain fast-speed records at 6 A.M., noon, 6 P.M., and midnight.*

(ii) Manual operation of the chart speed is provided on all records. Additional samples may be obtained by the operator during storm periods. The frequency of these samples and the number of the records analyzed is left to the discretion of the operator and analyst.

(b) Establishing the Point in Time of Readings:

(i) The beginning time should always be marked on the chart when a new roll is placed in operation, and also when it is removed from the recorder.

(ii) If possible, time checks should be made on the chart during the recording period together with supplementary remarks concerning the character of the surface waves.**

* The Sangamo Timer used for programming the frequency of fast runs and their duration may be used in a number of combinations.

** These periodic remarks should include time of observation, direction of waves, the stage of the tide, and descriptive remarks about the character of the water surface--such as calm, rough, white caps, etc.

(iii) A progressive time determination is made assuming six hour intervals between the beginnings of the fast-speed runs. The time at any point on the chart can be determined by measuring the chart length (or counting divisions) from the known time.*

(iv) The time of the reading is defined as the mean time of the interval chosen for analysis.

(c) Selection of Interval to be Analyzed Within the Fast-Speed Portion of the Chart:

The programming of the fast-speed portion of the wave record would logically be a direct function of the average period of waves during that portion. That is, the analysis is normally based on a given number of waves, and for equal numbers of waves measured per interval, one would select short and long fast-speed intervals for waves of shorter and longer periods, respectively. While this would be statistically consistent, it would require the impracticability of period forecasting and the inconvenience of variable programming, or the use of an excessive amount of chart paper. Hence the following plan is used, based on the analysis of a fixed time interval.

(i) Select the interval to include 20 minutes of the fast-speed portion and, if possible, select this interval such that its mid-point will approximately coincide with the mid-point of the fast-speed run. This assumes a fast-speed run to be approximately 25 minutes in length.

(ii) In the event that this interval cannot be taken (due to the end of the chart or variation in cam action of the programmer), select as great an interval as may be possible, centering the interval so that at least one minute is allowed at the beginning of the interval in order for the chart speed to reach its full speed (particularly if the spring wound recorder is used).

A typical wave record has been analyzed and reproduced here to illustrate the analysis procedure. Notice that the time of the beginning of the sampling interval is 0829 whereas the beginning time of the fast-speed run is at 0827. An interval of 20 minutes has been selected in this fast-speed portion, hence the end of the sampling interval is 0847.

The time of the interval being centered within this 20 minute interval is therefore, 0839.

* Fast chart speed corresponds to 3 inches per minute, slow chart speed corresponds to 3 inches per hour.

(d) Determination of the Significant Period:

(i) Having defined the sampling interval, the next step is to select several groups of waves, within the interval, that contain a series of well-defined waves.

(ii) Measure the length of time from the beginning to the end of each of the series of well-defined waves and count the number of waves included in this series.

(iii) Divide the sum of the time-intervals of the groups of waves by the total number of waves counted in all such groups.

Thus $T = t/n$

where t = total time interval between the beginning and end of all well-defined series of waves.

n = total number of waves included in all of the series.

T = the significant Wave Period

From the example, we see that there have been six such groups of waves selected and that the characteristic period of this interval is found to be:

$$T = n/t = \frac{44+87+193+75+103+114}{3+6+13+5+7+8} = \frac{616}{42} = \underline{\underline{14.7 \text{ sec}}}$$

(e) Determination of the Number of Significant Waves to be Measured:

(i) Divide the interval by the significant wave period to determine the number of waves within the interval.

For example:

$$\frac{\text{Interval (in seconds)}}{\text{Period (seconds per wave)}} = \text{N number of waves}$$

$$\frac{20 \times 60}{14.7} = \frac{1200}{14.7} = 82 \text{ waves}$$

(ii) Measure the highest N/3 waves $\frac{N}{3} = \frac{82}{3} \approx 27$ *

* In the example given, the highest 30 waves have been selected for measurement, and after measurement, the three lowest of this group have been omitted from the average. Normally, the analyst need select only 1/3 of the waves for measurement.

(a) Scan the record selecting the highest waves observed until N/3 waves are selected.

(b) Measure the height of the waves in divisions and record.

As may be seen on the sample analysis Data Sheet, the values of the wave heights have been recorded, the remaining part of the analysis being to arrive at the significant wave heights from these data.

(f) Determination of Significant Wave Heights:

(i) Determine the average of the highest N/3 waves as recorded (in this case 27 waves). Since the waves are measured in terms of the recorder chart divisions, the designation of this average is $R_{1/3}$. From the example, $R_{1/3} = 15.9$.*

(ii) Record the maximum wave height encountered $R_{max}$, in this case 19.5 divisions.

(3) Evaluation of Wave Heights from Chart-divisions to Wave Height (in feet-of-water):

(i) The following equation is used to obtain the surface wave height:

$$H = (C_i/K)\, R_i$$

where: H = wave height at surface, in feet.

$C_i$ = calibration factor of the instrument in $\frac{\text{Feet of Water Pressure}}{\text{Chart Divisions}}$ =

K = pressure response factor based upon depth of the instrument, depth of the water and length (or period) of wave being recorded.

$R_i$ = reading (in divisions) taken from chart as shown above.

(ii) Since the characteristic wave height and the maximum wave experience are the only two surface wave heights required in compiling the statistical information, only the values from the preceding determination need be used as the value of "R" in the above equation.

* Omitting 12, 12.5, 12.5 (three lowest values of the 30 selected for measurement).

Example:

$$H_{1/3} = \frac{R_{1/3}}{K} C$$

$$H_{1/3} = \frac{15.9}{0.822} 0.0755 = \underline{1.46 \text{ feet}}$$

$$H_{max} = \frac{19.5}{0.822} 0.0755 = \underline{1.79 \text{ feet}}$$

(iii) The value of $C_i$, 0.0755, was determined by laboratory test.

(iv) The values of K for various conditions of depth, period, etc. may be readily computed or obtained from prepared tables and graphs. (See sections IIB and C of this Manual, or References VI F-28 and 29)

(4) Recording the Statistical Information:

The purpose of analyzing wave data is to finally compile statistics that give a time history of the characteristics of surface waves. This information may be tabulated on a form as shown below and eventually may be graphed to a time scale.

CHARACTER OF THE SEA TABLE

AT _____ FROM _____ TO _____

| DAY | HOUR | T | $H_{1/3}$ | $H_{max}$ | REMARKS |
|---|---|---|---|---|---|
| | | | | | |

## SAMPLE ANALYSIS - DATA SHEET
## POINT SUR, CALIFORNIA

| | | | | |
|---|---|---|---|---|
| ROLL NUMBER | 273 | | | |
| DATE | May 11, 1950 | | | |
| TIME OF READING | 0839 | | | |
| PERIOD | 13.6 sec. | | | |
| N/3 | 27 | | | |
| WAVE NO. | WAVE HEIGHT - IN DIVISIONS | | | |
| 1 | 14 | | | |
| 2 | 13.5 | | | |
| 3 | 13 | | | |
| 4 | 12.5* | | | |
| 5 | 15 | | | |
| 6 | 17.5 | | | |
| 7 | 16 | | | |
| 8 | 17 | | | |
| 9 | 18 | | | |
| 10 | 19.5 | | | |
| 11 | 18 | | | |
| 12 | 14.5 | | | |
| 13 | 18 | | | |
| 14 | 15.5 | | | |
| 15 | 12.5* | | | |
| 16 | 12 * | | | |
| 17 | 17 | | | |
| 18 | 16.5 | | | |
| 19 | 18 | | | |
| 20 | 18.5 | | | |
| 21 | 13 | | | |
| 22 | 13 | | | |
| 23 | 17 | | | |
| 24 | 17 | | | |
| 25 | 16 | | | |
| 26 | 18.5 | | | |
| 27 | 14 | | | |
| 28 | 12.5 | | | |
| 29 | 15 | | | |
| 30 | 15 | | | |
| $R_{max}$ | 19.5 | | | |
| $R_{1/3}$ | 15.9 | | | |
| K | 0.822 | | | |
| $H_{1/3}$ | 1.46 Ft. | | | |
| $H_{max}$ | 1.79 Ft. | | | |

K, the ratio of the subsurface pressure to the surface wave height, was determined for a depth of water of 65 feet and for an average period of 14.7 sec.

* These values were not used.

5. References:

VI F-1 Admiralty Research Laboratory; "The Generation and Propagation of Ocean Waves and Swell"; Teddington, Middlesex, England; 27 February 1947.

VI F-2 Bascom, W.J.; "Installation of Wave Recorder at Heceta Head, Oregon"; Technical Report HE 116-259; Institute of Engineering Research, University of California; May 1947.

VI F-3 Bascom, W.J.; "Installation of Wave Recorders at Point Arguello, California"; Technical Report HE 116-291: Institute of Engineering Research, University of California; July 1948a.

VI F-4 Bascom, W.J.; "Installation of Wave Recorder at Quillayute, Washington"; Field Report No. 5; Institute of Engineering Research, University of California; October 1948b.

VI F-5 Bascom, W.J.; "Recording of Waves in Shallow Water at Sea"; Series 29 - Issue 27, Institute of Engineering Research, University of California; May 1950.

VI F-6 Beach Erosion Board; "An Ocean Wave Measuring Instrument", Technical Memorandum No. 6; Corps of Engineers, October, 1948.

VI F-7 Beach Erosion Board; "The Rayleigh Disk as a Wave Direction Indicator", Technical Memorandum No. 18; Corps of Engineers, July 1950.

VI F-8 Chinn, A. J.; "Summary Report on Shore Wave Recorder Mark III", Technical Report HE-116-303, University of California, Institute of Engineering Research; Berkeley, California; 21 June 1949.

VI F-9 Chinn, A. J. and Isaacs, J. D.; "The Testing and Calibration of the Mark III Shore Wave Recorders (Manufactured by the Mare Island Navy Yard)"; Technical Report HE-152-1, University of California, Institute of Engineering Research; Berkeley, California; 16 June 1948.

VI F-10 Folsom, R. G.; "Field Test of Shore Wave Recorders, Half Moon Bay"; Technical Report HE-116-282, University of California, Institute of Engineering Research; Berkeley, California; 15 January 1946.

VI F-11 Folsom, R. G.; "Subsurface Pressures Due to Oscillatory Waves"; Transactions of the American Geophysical Union, Vol. 28, No. 5; October 1947; pp. 722-724.

VI F-12 Folsom, R. G.; "Measurement of Ocean Waves"; Transactions of the American Geophysical Union, Vol. 30, No. 5; October 1949.

VI F-13 Isaacs, J. D. and Wiegel, R.L.; "The Thermopile Wave Meter"; Transactions of the American Geophysical Union, Vol. 31, No. 5; October 1950.

VI F-14 Klebba, Arthur A.; "A Summary of Shore Recording Wavemeters"; Woods Hole Oceanographic Institution; 3 August 1945.

VI F-15 Munk, W.H.; "Increase in the Period of Waves Traveling Over Large Distances: With Application to Tsunamis; Swell, and Seismic Surface Waves"; Transactions of the American Geophysical Union, Vol. 28, No. 2, pp. 198-217, April 1947.

VI F-16 Munk, W. H.; "Tracking Storms by Forerunners of Swell"; The Journal of Meteorology, Vol. 4, No. 2; pp. 45-57; April 1947.

VI F-17 Patrick, Lieut. (jg) D.A. and Wiegel, R.L.; "Portable Mark V Wave Recorder"; Series 29 - Issue 17; Institute of Engineering Research, University of California; January 1950.

VI F-18 Patrick, Lieut. (jg) D.A. and Wiegel, R.L.; "Breaker Height Measuring Devices"; Series 29 - Issue 18; Institute of Engineering Research, University of California; March 1950.

VI F-19 Putz, R. R.; "Wave Height Variability", Series 3 - Issue 318, University of California, Institute of Engineering Research; Berkeley, California; September 1950 (unpublished).

VI F-20 Rudnick, Philip; "A System for Recording and Analyzing Random Processes, University of California, Marine Physical Laboratory, San Diego, California; 5 October 1949.

VI F-21 Scripps Institution of Oceanography; "Proposed Uniform Procedure for Observing Waves and Interpreting Instrument Records"; Scripps Institution of Oceanography Wave Project, Report No. 26; 12 December 1944.

VI F-22 Seiwell, H.R.; "Investigation of Underwater Pressure Records and Simultaneous Sea Surface Patterns"; Transactions of the American Geophysical Union, Vol. 28, No. 5; pp. 722-724; October 1947.

VI F-23 Seiwell, H. R.; "Sea Surface Roughness Measurements in Theory and Practice"; Annals of the New York Academy of Science, Vol. 51, Article 3, pp. 483-500; May 13, 1949.

VI F-24 Seiwell, H. R. and Wadsworth, G.P.; "A New Development in Ocean Wave Research"; Science, Vol. 109, No. 2829, pp. 271-274; 18 March 1949.

VI F-25 Seiwell, H.R.; "The Principles of Time Series Analyses Applied to Ocean Wave Data"; Proceedings of the National Academy of Sciences, Vol. 35, No. 9, pp. 518-528; September 1949.

VI F-26 Snodgrass, F.E. and Stiling, D.E.; "Analysis of Wave Records"; Series 3, Issue 307, Institute of Engineering Research, University of California, Berkeley, California; January, 1951 (unpublished).

VI F-27 U.S. Naval Ordnance Laboratory; "The Wave Gage Installed at Cape Henry, Virginia"; U.S. Naval Ordnance Laboratory, NAVORD Report 1727; January 1951.

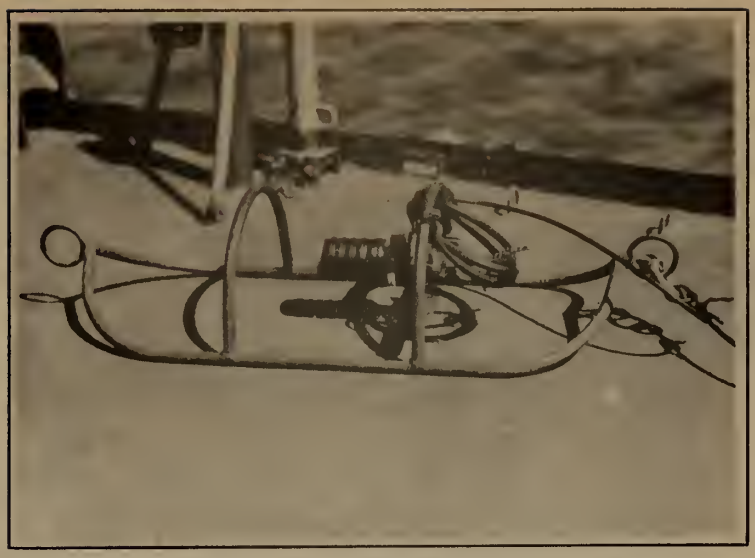
VI F-28 Wiegel, R.L.; "Tables of the Functions of d/L and $d/L_0$, Technical Report HE-116-265, University of California, Institute of Engineering Research; Berkeley, California; 30 January 1947.

VI F-29 Wiegel, R.L.; "Pressure Response at the Ocean Bottom Due to Surface Gravity Waves", Technical Report HE-116-108, University of California, Institute of Engineering Research; Berkeley, California; 8 September, 1948.

VI F-30 Wiegel, R.L.; "An Analysis of Data From Wave Recorders on the Pacific Coast of the United States", Technical Report HE-116-289, University of California, Institute of Engineering Research; Berkeley, California; 2 January 1949.

VI F-31 Wiegel, R.L.; "Installation of the Thermopile Wave Meter at Santa Margarita River Beach, Camp Del Mar, Oceanside, California"; Technical Report 155-2; Institute of Engineering Research, University of California; June 1949.

VI F-32 Woods Hole Oceanographic Institution; "A Bermuda Recording Installation"; Woods Hole Oceanographic Institution; Contract NObs-2083; August 1947.



a. Mark V thermopile pressure head in sled mounting (about 30" long). Demolition cable (2 conductor) is adequate for handling and hoisting.



b. Speedomax recorder, which makes ink trace of wave pressure on graph paper. Power is supplied by the LCP battery converted to 110-volt a.c. by converter at right.

FIG. VI F-1 - Temporary wave recorder installed by underwater demolition team

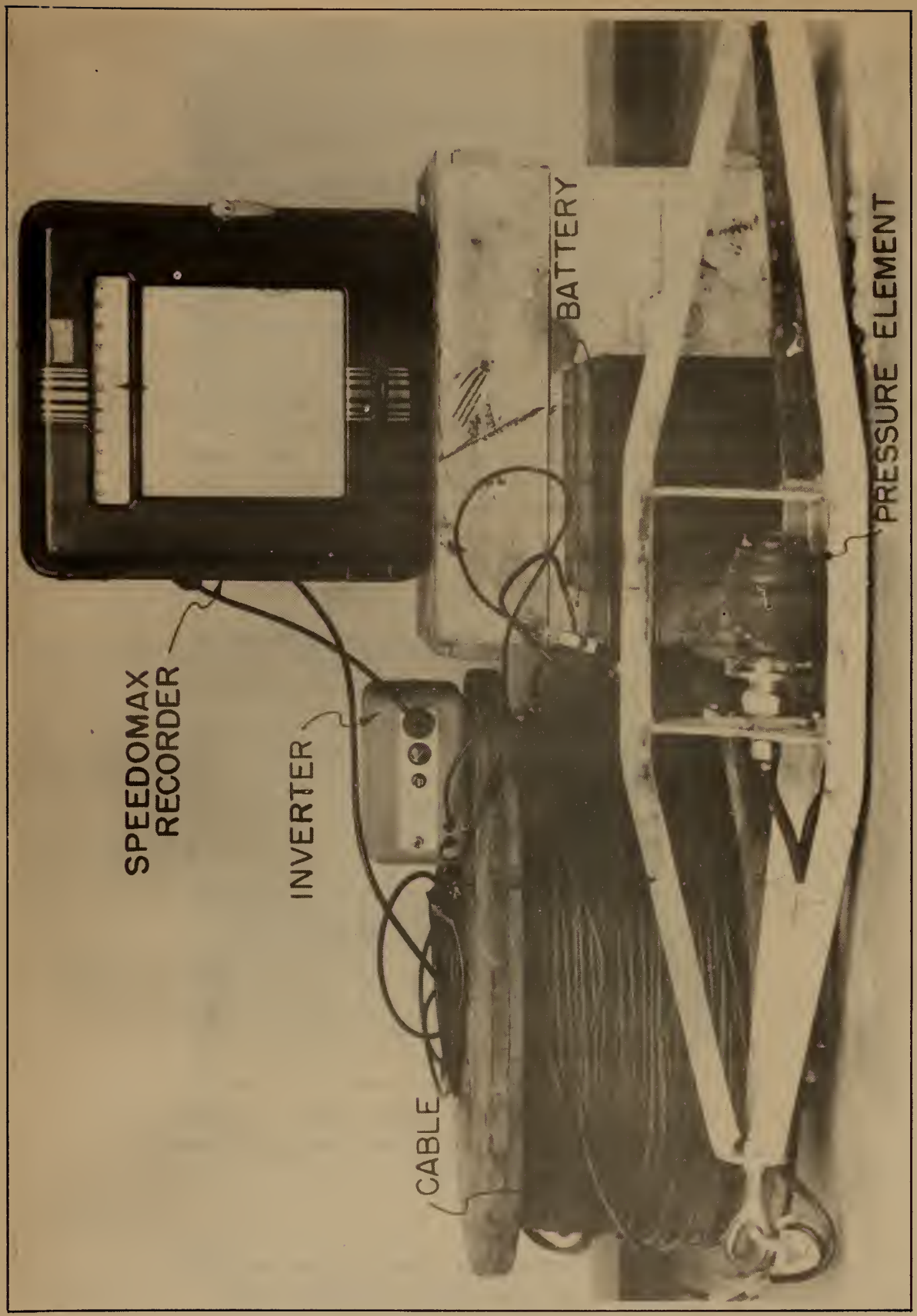



FIG. VII F-2 - Display view of a Mark V portable wave recorder



d. Setting poles in the surf zone at low tide.

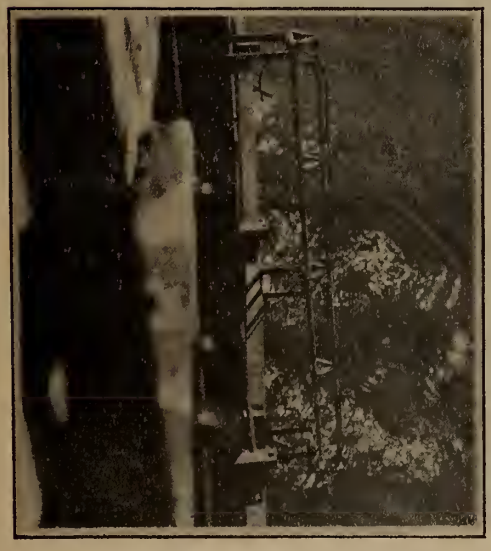


b. Sea sled with 20-foot surf pole being towed into position.

FIG. VII F-3 - Surf poles used to measure breaker height



a. Pressure head ready to be installed.



b. Pressure head installed; depth of pressure head is 7 feet below MLLW.



c. Close-up of pressure head.

FIG. VI F-4 - Pier installation of a pressure type wave recorder.



a. Woods Hole pressure gage, tripod and spar buoy.



b. Two tripods built by the University of California.

FIG. VIF-5 - Tripods for sub-surface pressure gages



a. Preparing to lower the tripod; spar buoy length = 30 feet.



b. Tripod in place. Tide stage: 4 feet above MLLW.

FIG. VIF-6 - Short-line spar buoy installation

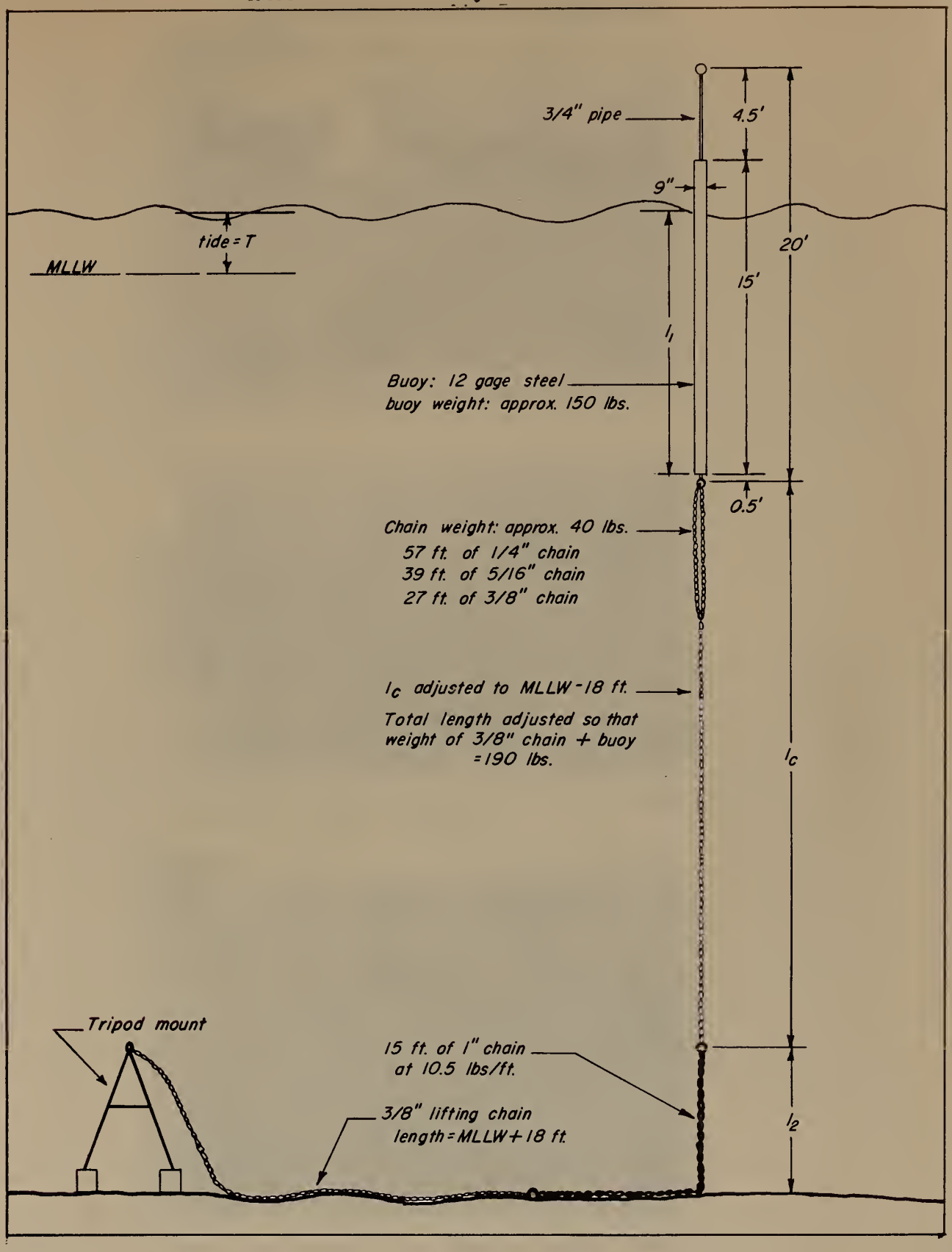



FIG. VI F-7 - Installation diagram for a spar buoy marker with chain anchor



a. 4-foot scarp, Oceanside, California, October 1949.



b. Shallow water reefs exposed at low tide, Cocoa, Florida.



c. Cable exposed on reef.

FIG. VIF-8 - Scarps and reefs



a. Spool holder mounting 2500 feet of armored cable.

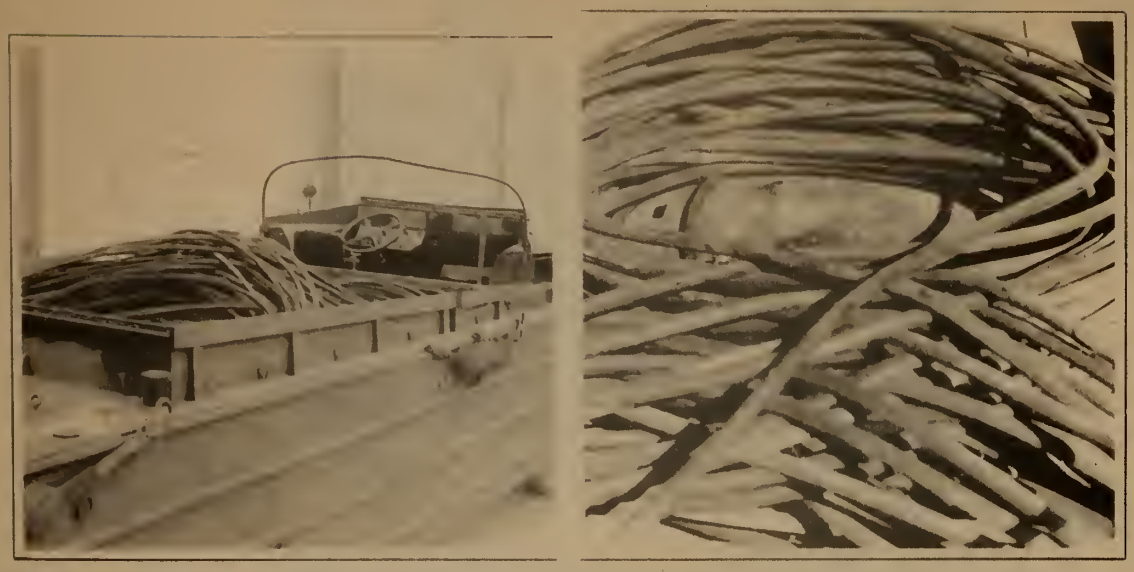


b. Spool holder mounted in a DUKW.

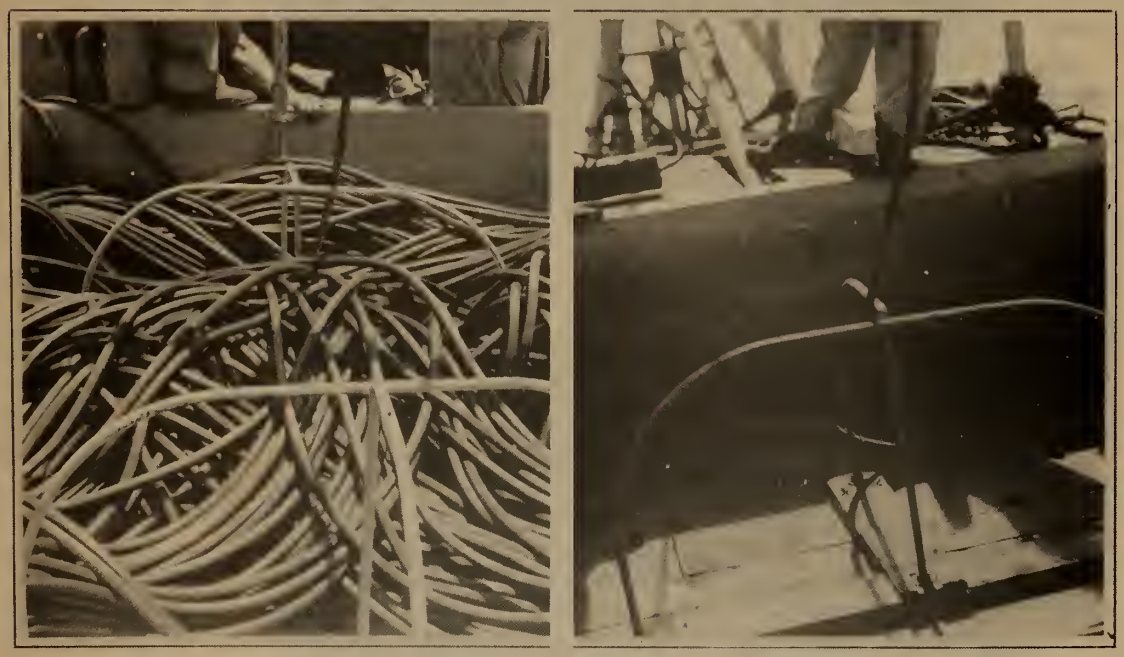


c. Spool holder mounted in an LCM.

FIG. VIF-9 - Cable spool holders



a. 2500 feet of armored cable faked in a DUKW.



b. 4000 feet of unarmored cable faked in a DUKW. Pipe framework installed to prevent fouling of cable.

FIG. VIF-10 - Cable faked in DUKW



FIG. VI F-11 - Installing cable at Pt. Arguello, California.



FIG. VI F-12 - Preparing to lay cable at Heceta Head, Oregon. All connections have been made; aircraft wing tanks between DUKWs were used to buoy the cable and reduce drag.

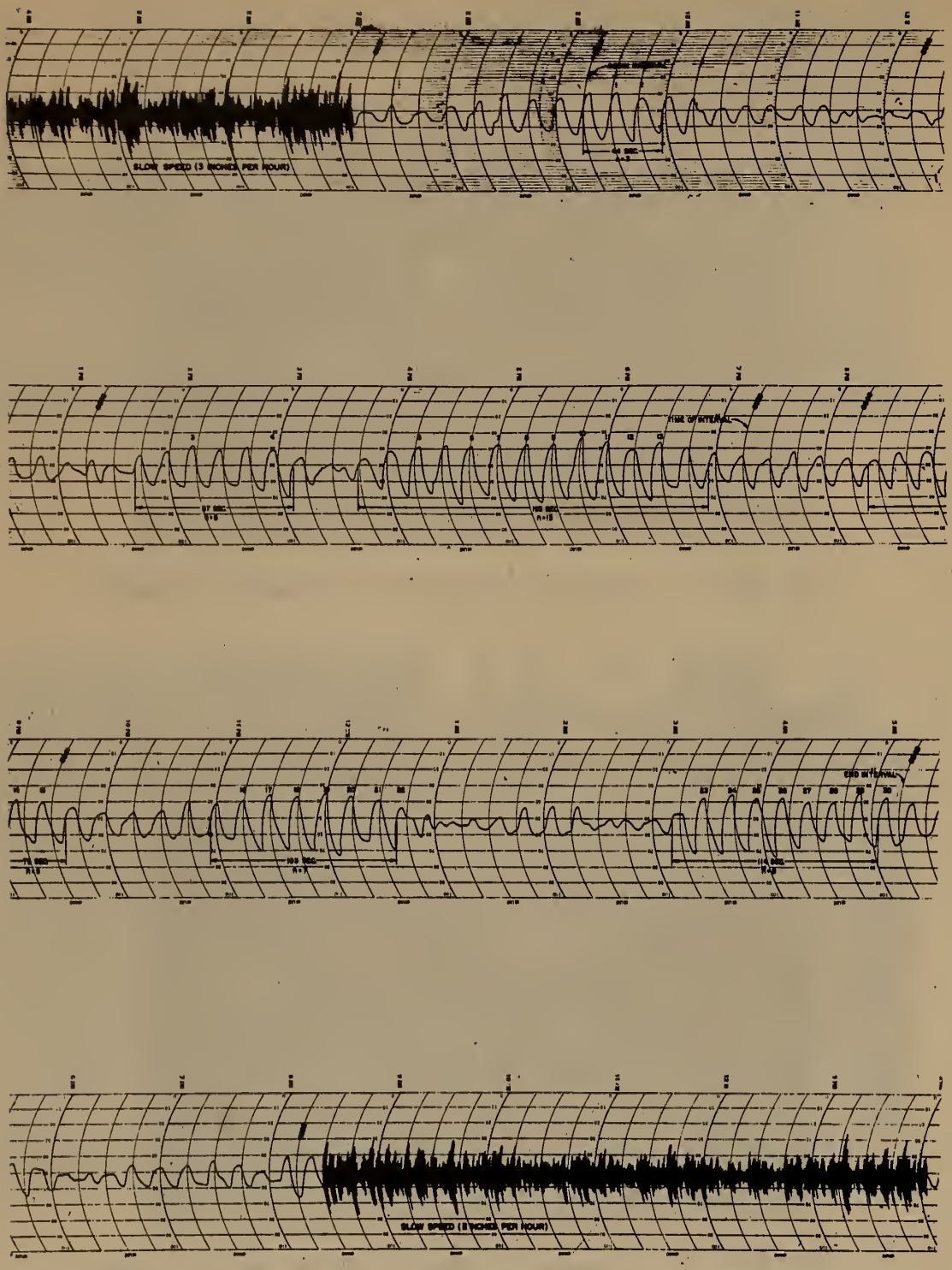



FIG. XIF-13 - SAMPLE WAVE RECORD ANALYSIS

MANUAL OF AMPHIBIOUS OCEANOGRAPHY

SECTION VI. INTELLIGENCE FOR AMPHIBIOUS OPERATIONS

## G. TIDE GAGES

BY
K. N. GRANTHEM

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# G. TIDE GAGES

## 1. Introduction:

The work of gauging the tide is carried out primarily by the Coast and Geodetic Survey. To date, several tide gages have been developed and used. They fall into two main types: Non-registering Tide Gages, and Self-registering Tide Gages. After a discussion of these gages, Tide Stations are briefly discussed in the last section.

## 2. Non-registering Tide Gages:

### a. Tide Staff:

The simplest kind of tide gage is a plain staff, which may consist of a board 1 to 2 inches thick and 4 to 6 inches wide, graduated into feet and tenths. This staff is fixed into place of such a length that both low and high tides can be recorded. This is called a "fixed staff".

### b. Portable Staff:

This is constructed so that it is easily removable when not actually measuring the tide (Figure VI G-1). The staff is removed and stored in a safe dry place protected from the weather. The portable tide gage can be kept clean and readable despite large amounts of oil and refuse in the harbor - which is naturally a great advantage. When this tide gage is in use, the gage support should be permanently fastened to a fixed reference point. With the gage support permanently fixed, the tide gage can be reassembled and its zero point will be at the same elevation after each assembly.

### c. Multiple Staff Tide Gage:

Along shore, where shoal water extends some distance offshore and the range of the tide is too large to be measured by a single staff, a succession of staffs may be used. The different staffs should be so graduated and installed that the graduations will be continuous from one staff to the next, with the readings on all of the staffs referred to the same zero.

### d. Tape Tide Gages:

Staff type gages (above) are primarily built for calm water. The tape tide gage is used where rough water is common (Figure VI G-2). The tape gage is operated by a float well which dampens out the larger wind waves. A tape, connected with the float, passes over a pulley that is attached to the ceiling of the tide house. The tape is kept taut by a counterweight.

There are two main types of tape gages: In one, a pointer attached to the tape moves over a fixed scale. In the other, there is a fixed reading mark on a board - and the height of the tides are read in relation to that fixed mark. The float from the tape gage must be cylindrical so that small changes in the floatation plane will not affect the cross-section area in this plane.

Caution is necessary in taking readings with a tape gage. A distinction must be made between a visible scale zero and the true datum of the gage - to which the observed heights are referred. The datum of the tape gage may be defined as the water level when the gage reading is zero. This datum will obviously be a number of feet lower than any visible scale zero inside the tide house.

e. Pipe Gages:

On offshore shoals, the pipe gage has been used successfully in depths up to 30 feet. Essential parts of a pipe gage are:

(i) A white pine rod staff (cross section 1 by 1 inch, with rounded edges graduated on all four sides in feet and two-tenths feet - with zero at the top;

(ii) One white pine float (1½ inches outside diameter, 7/8 inch inside diameter). This float should be thoroughly covered with shellac and liquid paraffin.

(iii) One 2-inch iron pipe (the bottom of which is set in a 1,000 pound concrete block which serves as an anchor). The pipe should be long enough to extend above ordinary high tides.

(iv) A top for the iron pipe, with a square hole for the staff to pass through.

(v) Additional equipment needed for anchorage and for keeping the gage vertical are: one 2-inch flange, 4 small sheaves, 4 guy wires (No. 6 wire), and 4 concrete blocks of about 2,000 pounds each.

f. Fathometer:

The fathometer operates by measuring sound waves travelling from the fixed origin to the ocean bottom and back. If the time interval is known (for the sound to make a round trip from origin to ocean bottom), it is possible to calculate the tide height. When using the fathometer, certain precautions should be taken: the ocean bottom should be as level as possible; the anchorage must be buoyed; and observations should be repeated on different days.

The best method of using the fathometer is to place the transceiver on the ocean bottom and send the sound wave to the surface. From there, the sound is echoed back to the transceiver below. As the time interval is known, the tide can be calculated. This method would not be affected by bottom irregularities at the ship's anchorage.

3. Self-registering Tide Gages:

a. Standard Automatic Tide Gages:

"The present standard automatic tide gage is a development of the Stierle gage. A float operates in a vertical box or pipe to which the slow moving tide has free access. The more rapid wind waves are largely damped out by the relatively small size of the inlet to the box. The rising and falling of the float operates a worm screw on the gage, which moves a pencil to and fro across a wide strip of paper which is moved forward by clockwork." (Figures VI G-3 and 4) "The combined motion of pencil and paper gives a continuous graph showing the rise and fall of the tide." (Manual of Tide Observations, VI G-5)

b. Portable Automatic Tide Gage:

This was designed primarily to obtain short series of observations for the reduction of soundings in hydrographic surveys. The portable automatic tide gage is smaller, and differs from the standard automatic tide gage in many ways.

This gage has a single roller with a single clock movement inside. Its float is smaller in proportion than the standard float, and it uses standard-sized iron pipe. This gage needs no specially built shelter as its iron cover protects it from weather and other damage. The record is drawn on regular cross-sectioned rolls and it is easily removed from the gage when desirable.

c. Pressure Gages:

These operate by measuring the pressure variations at the water bottom which are caused by the rise and fall of the tide.

During 1945, the University of California (Department of Engineering) developed a portable Recording Tide Gage. This pressure-time gage is mounted inside a watertight case. In normal usage, it is placed on the bottom of the ocean, to record the changes in mean sea level due to tidal variations. At all times, the depth of the water over the instrument must be greater than 5 feet, but less than 45 feet.

The recorder pressure element is connected to the external pressure through a loose rubber diaphragm and a series of felt disks. The diaphragm keeps the corrosive sea water out of the instrument parts; and the felt disks damp out the short period pressure fluctuations that are caused by waves.

A complete description of the instrument, and instructions for its operation are given in University of California Reports (Iversen, 1945a and b. See references VI G-2 and 3.) Figures VI G-5 and 6 illustrate the portable tide gage.

The maximum time for recording is about 48 hours. The record chart makes one complete revolution in 24 hours.

4. Tide Stations:

a. Primary Tide Stations:

These are permanent or semi-permanent stations that keep continuous records of the tides in any locality. These records constitute basic tidal data for present and future use. It is therefore most important that these stations be installed and maintained as efficiently as possible to insure the highest possible reliability and precision of their records.

The essential equipment of primary tide stations includes: an automatic tide gage, float well, shelter, tide staff, and a system of bench marks.

b. Secondary Tide Stations:

These are operated for a very limited period: usually less than a year, but more than a year in some cases. Secondary tide stations are established not only to obtain general tidal information for one locality, but also to obtain specific data for reduction of soundings in connection with the hydrographic surveys.

Although observations of secondary stations are usually insufficient for precise independent determination of tidal planes, they can give very good results when compared with the simultaneous observations at a good primary tide station.

5. Appendix:

a. References Cited:

VI G-1. Final Report on Wave Investigations, (1945), University of California, Institute of Engineering Research, (by Waves Investigation Staff), Technical Report HE-116-165, August 31, 1945.

VI G-2. Iversen, H. W., (1945a), Instructions for the Operation of University of California Tide Gage, University of California, Institute of Engineering Research, Technical Report HE-116-60, Feb. 18, 1945.

VI G-3. Iversen, H. W., (1945b), Instruction Book for Portable Tide Gage, University of California, Institute of Engineering Research, Technical Report HE-116-118, July 20, 1945.

VI G-4. Jones, E. Lester, Tide and Current Investigations of the Coast and Geodetic Survey.

VI G-5. Manual of Tide Observations, (1941), Special Publication No. 196, (Revised), U. S. Government Printing Office, Washington, D. C.



Fig. VI G-1. Portable Tide Staff and Support

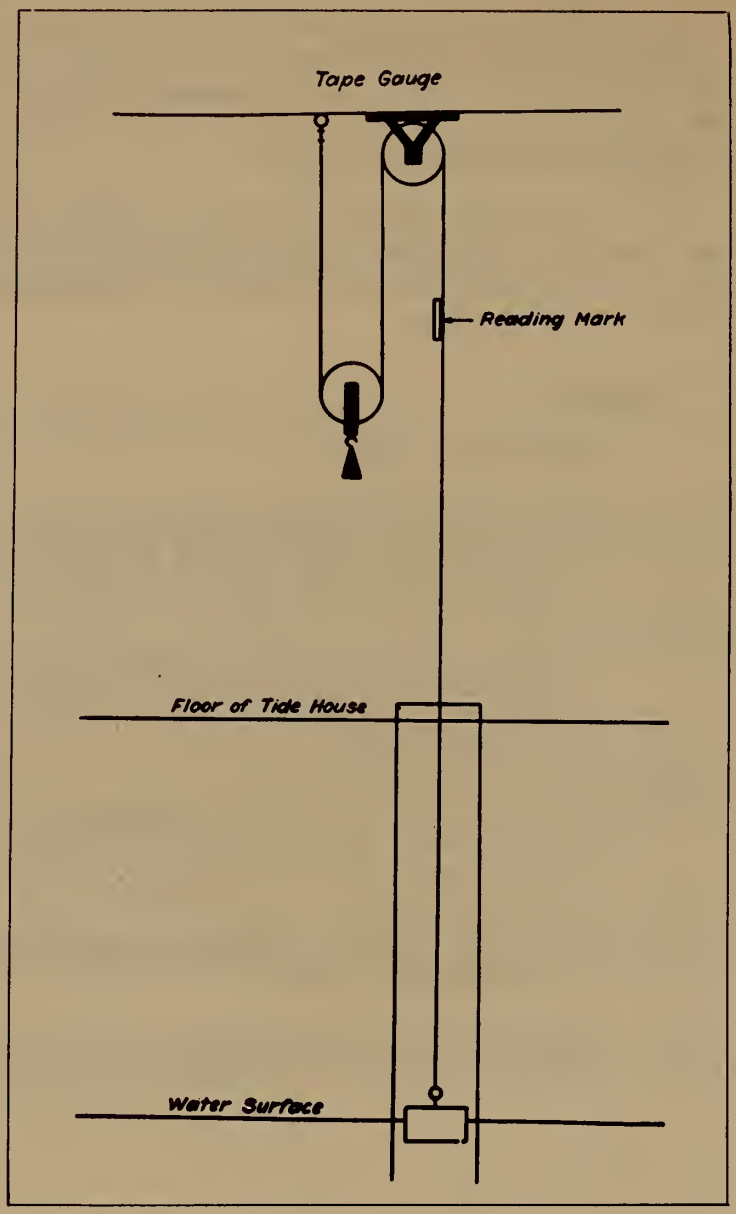



Fig. VIG-2. Tape Gauge

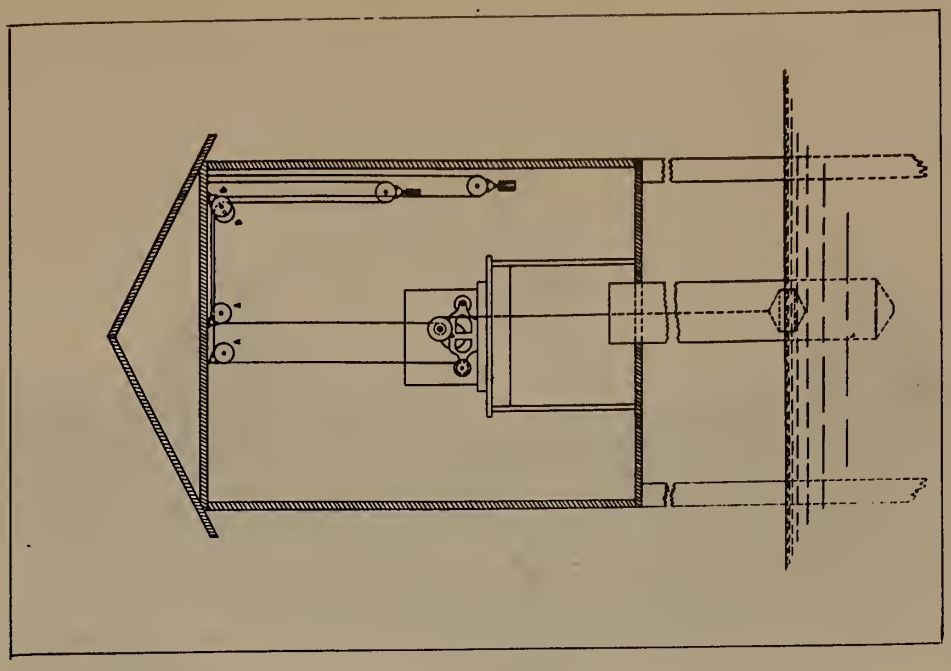


Fig. VI G-4. Schematic view of installation of Standard Automatic Tide Gauge

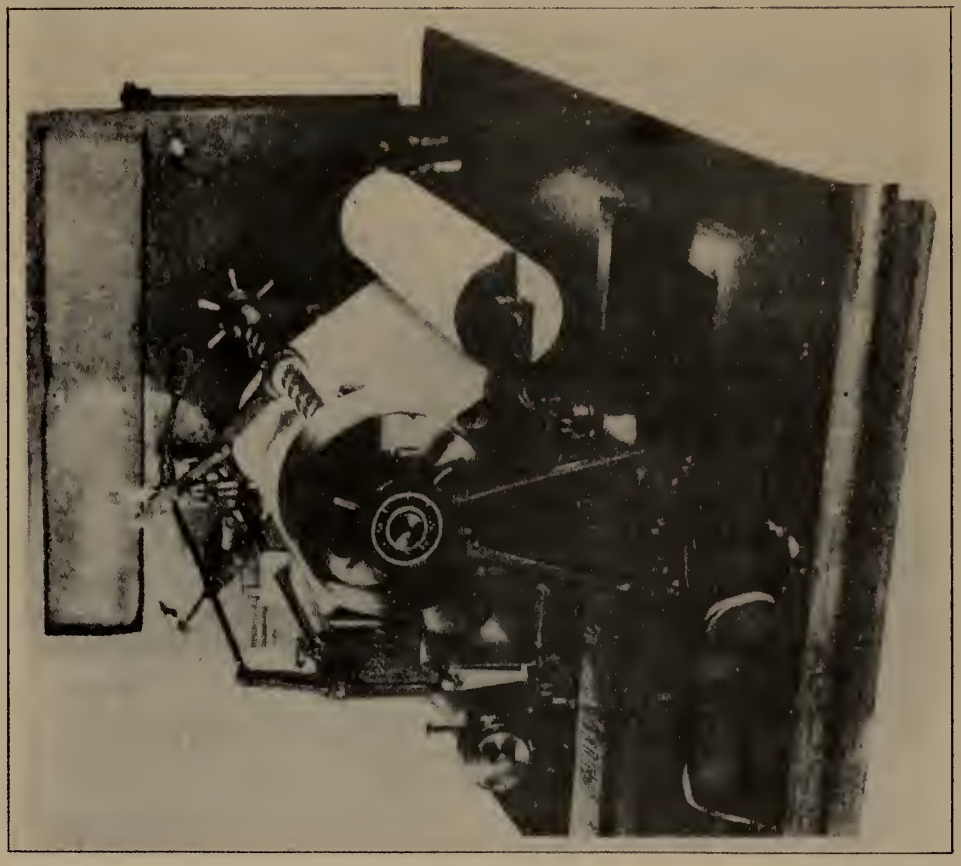


Fig. VI G-3. Standard Automatic Tide Gauge (paper installation)

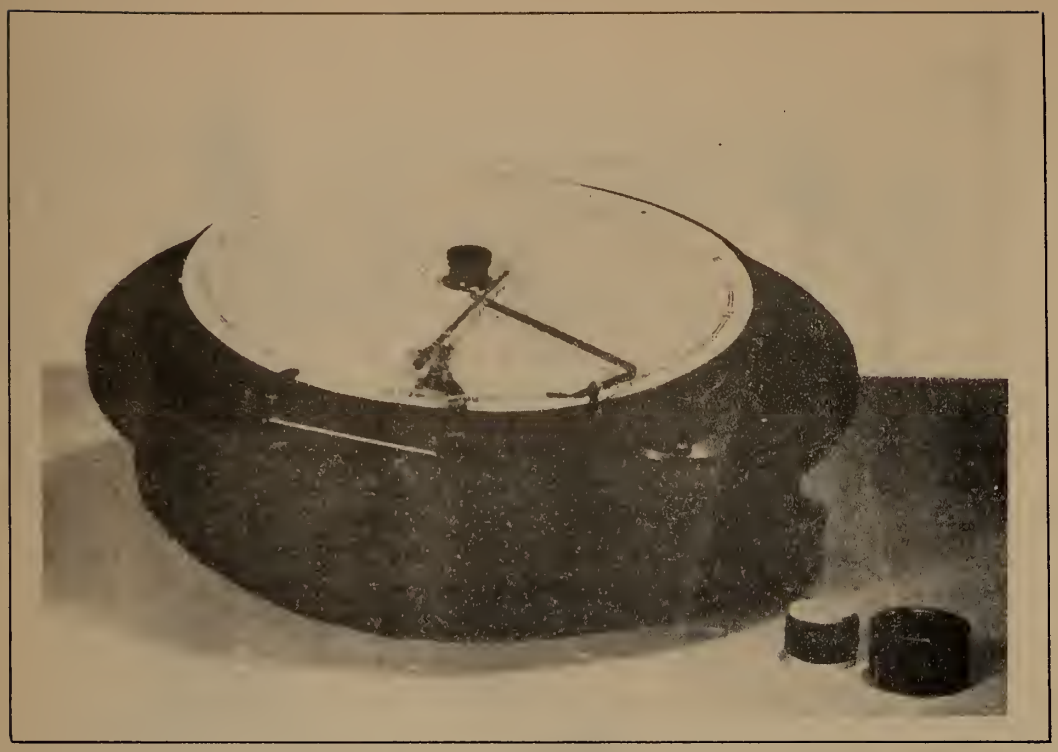


Fig. VI G-5. Portable Recording Tide Gauge



Fig. VI G-6. Loose bellows separating sea water and instrument parts

# MANUAL

# of

# AMPHIBIOUS OCEANOGRAPHY

## SECTION VII

## SURF CHARACTERISTICS AND SURFMANSHIP

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# MANUAL OF AMPHIBIOUS OCEANOGRAPHY

## SECTION VII: SURF CHARACTERISTICS AND SURFMANSHIP

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B. Landing Craft

C. Amphibian Vehicles.

D. Navy Lighter Pontoon Gear

E. Rubber Boats.

# MANUAL OF AMPHIBIOUS OCEANOGRAPHY

## SECTION VII. SURF CHARACTERISTICS AND SURFMANSHIP

### A. LANDING SHIPS

by
LTjg. D. A. Ellis, CEC, USN

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# A. LANDING SHIPS

## 1. Introduction:

Landing ships, such as the LSU, LSM, LCI and LST, are exposed to many combinations of sea, weather, and beach conditions. Sea and weather conditions which may be favorable at one landing site may prove hazardous at another landing site.

The purposes of this section are: to assemble all data pertinent to the beaching of landing ships; to correlate sea, weather, and beach conditions, and then to set forth beaching procedures and operational surf limits for the guidance of commanding officers of the various type ships. Unfortunately, there are not enough data on landing ships available to arrive at any but the most general conclusions for use in an amphibious landing.

Ocean waves break on a shoreline with varying degrees of intensity and ships react in many different ways depending on the waves and other factors. Because of this, the final decisions in a landing operation must be left to the judgment of the ship's commanding officers. The limited data assembled to date are presented here, in the hope that they may be of some value.

After considering three factors of importance in the landing of ships (breaker height, beach slope and longshore currents) some general rules for beaching and retracting are suggested.

## 2. Oceanographic Factors:

### a. Breaker Height:

Ideal breaker height seems to be less than 4 feet. In general, surf from 5 feet up is somewhat difficult, and every precaution should be observed in beaching. A dry ramp in surf over 5 feet high is practically unobtainable.

During tests in heavy surf at Ford Ord, California (Reference VII A-1) LSUs successfully landed and retracted in 14 foot breakers. It was felt that normal safe operational limits would be from 8 to 10 feet, however. It is assumed that other landing ships could operate in surf as great as that in which LSUs can operate.

### b. Importance of Beach Slope in Landing Operations:

The LST, LSM and LSU were designed to beach where the slope is 1 foot in 50 or steeper, and the steeper slope is preferable. In emergencies, these ships can partially counteract the difficulty of flatter slopes by proper use of ballast. When this is done, however, they usually have to ground farther out from shore, which results in a wet ramp.

### c. Importance of Longshore Currents in Landing Operations:

Ordinary longshore currents have little effect in beaching larger landing ships because of stern anchors, but the ship may have to be brought in at an angle in order to place its bow squarely on the beach. Otherwise, broaching may occur.

The record shows that LSTs can beach with longshore currents up to about $4\frac{1}{2}$ to 5 knots.

The bow cannot be readily beached in causeway marrying operations. In these operations, the bow anchor on the port side can be used to counteract the current from the port side. However, the LST and LSM cannot use the bow anchor

with current from the starboard side, as the chain may get foul of the doors or possibly puncture the ship's bottom.

3. General Rules for Beaching and Retracting of LSTs:

In planning a particular beaching operation, it is important to obtain all possible information about the beach that has been selected.

Important beach factors to be checked in advance are:

(i) slope of beach
(ii) character of bottom - whether sandy, rocky, etc.
(iii) approximate contour of bottom
(iv) rise and fall of tide
(v) predicted surf conditions
(vi) unusual features, such as obstructions (coral heads etc.)

When all the available information concerning the beach has been carefully studied, the decision can be made as to the amount and kind of cargo to be carried and the necessary draft and trim in landing condition. The LST must be trimmed so that the bow will ground first. If the beach is relatively flat, the depth of water over the ramp end may be too great for vehicles to be disembarked, and a pontoon causeway or other device must be employed to land them in shallower water.

At least 12 hours before the hour set for actual beaching, final preparations for beaching should be started. These preparations cannot be described in detail as they will vary to suit conditions and will generally be covered by instructions issued by the forces afloat. Some of the principal steps of the operation are as follows (from U.S. Navy BuShips 1945 VII A-2):

(i) Remove all dunnage, tank shoring, etc. from tank deck. The tanks will then be held by chains alone.

(ii) Turn over all tank engines to insure that they will start. Check ventilation of tank deck while engines are being tested. If any tank is out of commission, make arrangements for getting it clear of the others or for towing it onto the beach. Top-off gasoline in tanks.

(iii) About 3 hours before arrival, start pumping ballast out to bring ship to proper trim. About 2½ hours are required to empty the ballast tanks.

(iv) About 2 hours before arrival, remove the tank lashing chains, clear the tank deck and test lights and ventilation.

(v) Thirty to forty-five minutes before landing, start all ventilation fans; make sure they are operating at full capacity. Begin warming up tanks.

(vi) About 10 minutes before actual beaching, start opening the bow doors and be sure they are wide open (about 104° from the centerline of the ship).

If an LCT is carried on the main deck, or pontoons are carried on the sides, these should be launched prior to beaching at an appropriate distance offshore depending on weather conditions, character of beach, and enemy activity. LCVPs are also ordinarily launched prior to beaching.

The actual beaching operation can be satisfactorily accomplished with little difficulty if the beach is properly selected in advance and has reasonably good characteristics. The operation, however, does require proper training and adherence to certain fundamental principles. It has been found by a good deal of experience on various kinds of beaches that LSTs can be beached at speeds of from 4 to 10 knots, but speeds of 6 to 8 knots are usually the best beaching speeds. About 200 yards from the estimated grounding point, the stern anchor should be dropped and at least 600 feet of cable payed out just fast enough so that the anchor will not retard the speed of the ship. Both screws should be kept turning over at about one-third speed ahead (or from one-third to two-thirds speed) depending on judgment and experience. The ship will ground easily and firmly and, with a good sandy beach, will run up so as to decrease the draft forward anywhere from 6 - 18 inches, depending on the slope and character of the beach and the speed of approach. The ramp should then be lowered after making sure that the ship is hard and fast on the beach. There should be sufficient tension in the stern cable to keep the ship from swinging. The operation should be timed so that beaching will occur about an hour before high water slack, which provides the maximum length of time before the change in tide appreciably affects the ship. The importance of this timing will, of course, depend on the amount of rise and fall of tide. Unless the ship is to be retracted immediately after landing the tanks, it is important to fill forward ballast tanks to hold the ship firmly on the beach during unloading operations, and until retraction.

As the first few tanks land they will probably dig a hole just ahead of the ramp if the beach is sandy. A special mat has been developed to span or to prevent the formation of such a hole.

If the landing is made as recommended just before high water slack, retraction should be accomplished within an hour after high water slack or else it will be necessary to wait until the next high tide. With the height of tide approximately equal to that at the time of landing, the first step in retraction is to pump out any ballast water that may have been added forward, in order to lighten the bow. The ship will then be lighter than in the landing condition by the amount of cargo moved ashore. Under this condition it should be quite simple to retract the ship by taking a strain on the stern anchor and backing the engines. If, however, it becomes necessary to retract with a load approximating the load in landing condition, this operation will be somewhat more difficult. It can be done, however, if the height of tide is the same as at landing, by backing with full power on the engines, moving the stern with the rudders, and hauling in on the stern cable. In difficult cases it may help to run the engines full speed ahead for a short interval and then back at full speed to wash the sand out from under the ship.

In either the beaching or the retracting operation, it is extremely important to keep the ship headed perpendicular to the beach line. This is particularly true where there is an appreciable surf running. If the ship is beached perpendicular to the line of the surf, it will form a breakwater which will provide a good shelter from the surf in way of the ramp. On the other hand, if the ship is allowed to broach - that is, to swing toward a line parallel to the surf - salvage operations may be necessary to recover the ship. The screws should be used to assist the rudders in steering to avoid broaching. This is particularly important in backing off the beach.

4. General Rules for Other Landing Ships:

The procedures for beaching and retraction of the smaller landing ships employ the same general principles as outlined above for LSTs. Operational Information Publications are available as part of the ship's library, and these should be consulted for additional information.

5. Appendix:

a. References:

VII A-1: Bascom, W. N. - Test of Landing Craft at Monterey Bay - Series 29, Issue 20, Institute of Engineering Research, University of California, Berkeley, California, April 1950, (unpublished). RESTRICTED.

VII A-2: U.S. Navy, BuShips - LST Operational Information - Navships 250-420, January 1945. RESTRICTED.

VII A-3: U.S. Navy, Photographic Interpretation Center - Underwater Depth Determination - PIC Report 46, October 1944, pp 9 and 10. CONFIDENTIAL.

b. TABLE VII A-I BEACHING CHARACTERISTICS OF LANDING SHIPS AND CRAFT

The following table of characteristics is simplified to include only those aspects of craft critical to beaching. These are the characteristics that determine whether or not a craft is suitable for use on a given beach. Only major operational types of craft are listed.

| TYPE OF CRAFT | FULL DESIGNATION | LENGTH OF CRAFT | BEACHING DRAFT | | BOTTOM SLOPE OF CRAFT |
|---|---|---|---|---|---|
| | | | FORWARD | AFT | |
| LCA | Landing Craft (Assault) | 41' - 6" | 1' - 9" | 2' - 3" | 1:51 |
| LCVP | Landing Craft (Vehicle, Personnel) | 36' - 0" | 2' - 2" | 3' - 0" | 1:40 |
| LCP(R) | Landing Craft Personnel (ramp) | 36' - 8" | ------ | 3' - 6" | 1:100 |
| LCP(L) | Landing Craft Personnel (large) | 36' - 8" | ------ | 3' - 6" | 1:100 |
| LCS(S)2 | Landing Craft Support (small) mark 2 | 36' - 8" | 3' - 6" (max) | | Capable of Beaching |
| LCS(L)3 | Landing Craft Support (large) mark 3 | 158' - 5" | 4' - 6" | 5' - 10" | 1:120 |
| LCM(3) | Landing Craft Mechanized (mark 3) | 50' - 0" | 3' - 0" | 4' - 0" | 1:50 |
| LCC(2) | Landing Craft Control (mark 2) | 56' - 0" | 3' -11" (max) | | Not intended to beach |
| PGM* | Motor Gunboat | 110' - 0" | 6' - 9" (max) | | Not intended to beach |
| LSU(5) | Landing Craft Tank (mark 5) | 117' - 6" | 2' -10" | 4' - 2" | 1:105 |
| LSU(6) | Landing Craft Tank (mark 6) | 120' -5" | 3' - 4" | 4' - 0" | 1:200 |
| LCI(L) | Landing Craft Infantry (large) Nos. 1 to 350 | 158' - 5" | 2' - 8" | 4' -10" | 1:71 |
| LCI(L) | New Model | 158' - 5" | 3' - 0" | 5' - 0" | 1:79 |
| LSM | Landing Ship Medium | 203' - 6" | 3' - 5" | 6' -11¼" | 1:58 |
| *LST | Landing Ship Tank | 328' - 0" | 3' - 1" | 9' - 6" | 1:50 |
| APD | Converted "Flushdeck" Destroyers for landing raiding parties | 314' - 4" | 11' -11" (max) 4' sound dome | | Not intended to beach |

*All new models are rigged to carry 2 sections of standard pontoon causeway. Each causeway section is 175 feet long; light draft 18 inches. Older model LSTs can carry same equipment but special rigging is required.

Note on Anchorage Depths: All LCTs, LCIs, LSTs, and LSMs carry 150 fathoms of anchor cable or chain. Maximum good anchorage for these is 50 fathoms. Ground tackle on craft smaller than LCT is not standardized.

Photographic Interpretation Center - Report #46: Underwater Depth Determination Oct. 1944 - [redacted] (Reference VII A-3).



FIGURE VIIIA-1 - Vehicles being unloaded from LST via pontoon causeway



FIGURE VIIIA-2 - Three LSTs hard-grounded in deep water. It was necessary to ferry men and equipment ashore in DUKWs. 1246, 11/25/46






FIGURE VII'A-3 - THE LSM






FIGURE VII A-4 - THE LSV

SECTION VII. SURF CHARACTERISTICS AND SURFMANSHIP

## B. LANDING CRAFT

by

R. L. Wiegel and LTjg. D. A. Ellis, CEC, USN

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## B. LANDING CRAFT

### 1. Introduction:

Observation and analysis of many landing-craft sorties during amphibious training exercises has led to the conclusion that LCM and LCVP casualties* due to surf and beach features are:

(i) Broaching on the beach or on a bar.

(ii) Hanging up on a bar.

(iii) Swamping by breakers when landing or retracting.

This section discusses the causes of these casualties, and gives suggestions for decreasing the number of casualties. The information pertaining to the small landing craft (LCMs and LCVPs) is largely based upon observations of 545 LCVP and 633 LCM landings at Morro Bay, California; 4816 LCVP and 5258 LCM landings at Coronado, California; and 26 LCVP and 116 LCM landings at Fort Ord, California.

### 2. Factors Affecting Landing Craft Operation:

As a result of these studies it was concluded that:

> "Excluding mechanical failures occurring outside the surf zone and not caused by previous landings, all other casualties probably have surf as the ultimate cause. An experienced coxswain could land and retract without difficulty in fog or moderately high wind, and mechanical failures would occur much less frequently were it not for the existence of the surf. The number of variables is so great and their nature so intangible that it seems desirable to ultimately classify the casualties under the three principal headings Beach, Craft and Surf, and allow the scatter of the data to represent the effect of the other variables."
>
> (From Reference VII B-3)

#### a. Oceanographic Characteristics Affecting Operation:

The following oceanographic factors are most important in the operation of small landing craft:

(i) Breaker characteristics (type, height, period, and angle with beach)

(ii) Configuration of the beach

(iii) Currents

(iv) Tide

#### b. Craft Characteristics Affecting Operations:

In addition to these oceanographic characteristics, the characteristics of the craft must also be considered. For example, the chief cause of casualties to the present covered LVTs is their being pitched over by large breakers while

---

* Note: defined for this Manual as any mishap which temporarily or permanently prevents a craft from fulfilling its mission.

approaching the beach. On the other hand, LCVP and LCM casualties are mostly broachings (occurring at the beach face while the craft are being unloaded), with the second most common type of casualty that of the craft becoming "hung-up" on bars. In the series of observations mentioned in the introduction, it was found that in no instance was swamping in the surf the primary cause of a casualty. If swamping occurred, it happened after a craft had gotten into trouble because of a mechanical defect.

The following table gives an example of the relative importance of the types of casualties for one beach (Coronado, California):

TABLE VII B-1

Overall Percentages of Casualties for Coronado

| Type of Craft | Broaching, due to: | | | Hung-up on bar | Caused By Mechanical Defects |
|---|---|---|---|---|---|
| | Poor Seamanship | Surf conditions on the beach | Surf conditions on bars off the beach | | |
| LCVP | 2.2 | 2.9 | 0.3 | 0.8 | 0.4 |
| LCM | 1.9 | 1.0 | 0.1 | 0.3 | 0.2 |

(From Reference VII B-4)

The effect of the poor maneuverability of the single screw LCVP, compared with the twin-screw LCM, is apparent. LCVPs are much more liable to broach on a beach and also to become hung-up on a bar.

Another craft characteristic is the velocity. The top speed of both LCVPs and LCMs is 10 knots. It can be seen from Figure VII B-2 that breakers higher than 8 feet travel faster than the craft. Hence, the craft cannot "outrun" them while approaching the beach.

3. Oceanographic Factors:

a. Effect of Breakers:

(1) Type of Breaker: The height of the breakers is of major importance in landing operations, but other factors must also be considered before any absolute limits can be set. One of these is the type of breaker. Breakers are classified as spilling, plunging and surging; and operational limits for the various types of craft vary according to the type of breaker encountered. For LCVPs and LCMs, observations have shown that the plunging breaker is the most dangerous while landing or retracting, but that the surging type breaker is the most dangerous during the period the craft is "on the beach". (Figure VII B-3)

(2) Height of Breakers: As a result of tests by skilled personnel, and discussions with the crews and experienced boat officers (Reference VII B-1), certain conclusions were reached as to limiting heights of breakers for craft operation:

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- LCVPs could beach in surf two-thirds as high as could LCMs.

- LCVPs could retract through surf only one-half as high as could LCMs.

- LCVPs should not be used when loaded in surf higher than 5 to 6 feet except in emergencies. When their crafts were unloaded, highly trained personnel have been able to operate in breakers as high as 9 feet when landing, and 7 feet when retracting.

- LCMs should not be used - when loaded - in breakers higher than 8 feet, except in emergencies. LCMs when unloaded have been operated in breakers as high as 12 to 14 feet (Figure VII B-6).

It is doubtful (Reference VII B-1) whether LCMs retract through the larger breakers. Although they retract into the larger ones, it seems unlikely that they continue moving seaward in breakers over 8 feet high. It is more likely - when they encounter several larger breakers - that they simply stand still until a few smaller waves break, and then manage to traverse the surf. This theory is apparently justified by Figure VII B-7. The higher the breakers encountered, the longer the time necessary to retract (the width of the surf zone was approximately the same in all cases).

(3) Period of Breakers: Long period waves make the task of the coxswain easier, as they give him an opportunity to recover control of his craft between breakers. Also, for a given width of surf, fewer breakers are encountered. Short period waves, particularly on a flat beach with a wide surf zone, will swamp a disabled craft more quickly than long period waves of the same height.

(4) Breaker Angle: Waves breaking at an angle to the beach produce two hazards to craft operation. First, such waves set up littoral currents parallel to the shore which tend to broach crafts. Secondly, as coxswains tend to land or retract along a line perpendicular to the shore, waves (breaking at an angle to the beach) are also at an angle to the craft, which again tends to broach craft.

b. Configuration of the Beach:

Each type of beach and each type of surf imposes special problems on landing craft operation. An operation which may be carried out safely on one beach, with waves from a given storm breaking on it, may be hazardous upon a beach of another type. For example, waves with given deep-water dimensions (if unrefracted or if the refraction effects are the same) will develop into higher breakers on a steep beach than on a flat beach. See Figure VII B-8 for some examples of beaches and surf conditions.

(1) Mild and Flat Beaches*: (slope of 1/60 - 1/120; and less than 1/120) have wide surf zones with many lines of foam inside the breakers. Retraction from such beaches is slow and difficult. When the waves are high, the long time required for passing through the surf zone of a flat or mild beach

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* There are 5 classes of beaches, in regard to slope; steep, moderate, gentle, mild, and flat. In regard to small craft operation, mild and flat beaches may be considered together.

imposes a severe strain upon craft and personnel. Spilling breakers are the general rule, although very large plunging breakers may be encountered. These beaches usually have several lines of irregular bars, with waves breaking on outer bars and reforming and breaking again on inner bars.

Currents are often quite strong (several knots) in the troughs between the bars.

(2) Gentle Beaches: (slope of 1/30 - 1/60) have medium-width surf zones. They are less difficult to traverse than either the flatter beaches or the steeper beaches. There are usually two bars present during periods of steep storm waves. Beaches such as this may have the characteristics of either a flat beach or a steep beach, depending upon the tide stage. At high tide the beach may have the characteristics of a steep beach, while at low tide it may have the characteristics of a flat beach.

(3) Moderate Beaches: (slope of 1/15 - 1/30) normally have one bar. Often, two lines of breakers are present, the higher waves breaking on the bar and the lower waves breaking on the shore face. The waves are predominantly of the plunging type.

The height of the tide is an important factor when landing on a beach like this. At low tide, the offshore bar may make landing impossible, while at high tide the depth of water over the bar may be adequate for small craft.

Such beaches often have surf-rips which cut channels through the bars. Operations at Fort Ord, California (Reference VII B-1) have shown that LCMs and LCVPs can land and retract successfully in these channels during low tide while other craft, 20 feet away, will get hung-up on the bar.

(4) On Steep Beaches: (slope steeper than 1/15) waves usually break right on the beach face. Plunging breakers are the rule. However, surging breakers occur when long-period waves (15 seconds or so) break on very steep beaches (1/5 to 1/10). This has been found to be the most dangerous combination for LCMs and LCVPs (Reference VII B-7) because the full force of the wave hits the stern of the craft while it is trying to maintain its position on the beach while discharging personnel or cargo. The surging breakers swing the craft violently to one side, broaching it, and soon swamping it. (Figure VII B-3)

c. Currents:

Two types of currents are often present along beaches. One is the littoral (parallel to shore) current which is set up by waves breaking at an angle with the beach, and the other is the surf-rip.

Littoral currents are a frequent cause of broaching, particularly in conjunction with troughs landward of bars. On a steep beach, the current is not of great importance as it is the force of the breaker uprush which tends to broach craft. On a flat beach, or where there is a bar, a light current can broach a boat that gets its stern stuck while retracting. Figure VII B-9 (Reference VII B-2) shows examples of the cause and effect of littoral currents.

Surf-rips are caused by water piling up on the beach, then flowing parallel to the beach and out to sea. The currents are often strong. They cut channels through the bars which are very useful to small craft operators as, during low tide, they often furnish the only means of reaching the beach. Figure VII B-10 shows an example of such a rip current (from Reference VII B-2).

d. Tide:

The effect of the tide is complex, depending upon its range, the configuration of the beach (especially water depths over bars), beach trafficability, enemy or natural obstacles, etc. No general rules can be formulated for the most desirable state of the tide for amphibious landings.

For example, for LCVPs and LCMs it might appear that high tide is best as it gives them the best chance of clearing bars. They must discharge their cargo, however. Since the trafficability is poorest on the high, dry part of the beach, it usually takes longer to discharge wheeled vehicles at high tide than at low. The result is that the craft must remain at the beach longer, with more chance of broaching.

4. Handling Small Craft in the Surf:

a. Landing:

"The approach to the beach and the actual landing are the simplest phase of an amphibious operation*. When the surf is heavy, the boat should ride in on the back of a crest until the latter has broken. Then the coxswain gives it full throttle, and runs the boat as far up the beach as possible before dropping the ramp. The only danger in this phase of the operation is for the boat to get in the forward slope of a large wave and start tobogganing.** When this happens, the coxswain may lose control, and the boat may run off to the side, causing collisions, and sometimes capsizing. An alert coxswain can avoid this by backing down when he feels the wave approaching, and allowing it to go past him."
(Reference VII B-5)

In case bars are impassable at low tide, channels through the bars should be used if any are present.

b. On the Beach:

"As soon as the boat has beached, broaching lines, attached either to the bow*** or stern, are thrown out and the beach party uses them to try and keep the boat from broaching. The coxswain assists by using proper power to hold the bow hard against the beach. For any given condition, the amount of broaching will depend on the length of time that the boat is beached. The first waves of boats unloading troops seldom have much trouble,**** because the actual unloading takes less than a minute and they can get their ramps up and get out in a hurry. Boats in later waves, particularly those carrying loose cargo, may take ten to to fifteen minutes to unload, and during such a long period even a moderate surf may work them around and broach them."
(Reference VII B-5)

---

* Note: This applies to LCMs and LCVPs.
** Note: Surfboarding.
*** Note: Broaching lines to the bow are practically useless unless stern is grounded and bow is free.
**** Note: Except with surging breakers on steep beaches or when the ramp won't come up again.

The following are a few pointers which, if followed, should decrease casualties:

(i) The longer a boat is at the beach discharging men and equipment, the longer it is in danger of broaching. Get the ramp down, the cargo discharged, the ramp up and retract as fast as possible.

(ii) Don't wait for a craft to broach; if stern-broaching lines are used as soon as a craft lands, the casualties due to broaching will be greatly reduced.

(iii) If a craft broaches, make use of a stern line, not a bow line. Use a DUKW or a tractor to pull on it. Men can do little. Concentrate on craft just starting to broach; to work on a fully broached craft that is aground is a waste of valuable men and time if others can be righted in a short time.

(iv) LCVP and LCM coxswains should use power to keep the bow hard against the beach. On steep beaches with very small surging breakers (one to two feet high) there have been casualties because the craft (without power) surged a few feet seaward with the ramp down and soon filled with water and sank.

(v) If a ramp sticks while down, use a bulldozer to get it up quickly; then retract before you broach. Do not retract unless the ramp is dogged down.

c. Retracting:

"Boats must be backed off the beach until beyond the breaker line. If an attempt is made to turn the boat inside the breakers, it may be caught broadside by a wave and either be swamped or capsized. Also, heavy surf has been known to stave in the ramps of boats heading out bow first. During retraction, the boat is bound to ship a certain amount of water and the coxswain must keep this to a minimum by slowing down just before hitting a wave, and letting it run past him. The criterion used to determine whether a landing is possible is the ability of the boats to get off the beach without broaching, and to retract through the surf without swamping."
(Reference VII B-5).

5. Appendix:

a. References:

VII B-1: Bascom, W. N. - Tests of Landing Craft at Monterey Bay - Series 29, Issue 20, Institute of Engineering Research, University of California, Berkeley, California, April 1950, (unpublished), RESTRICTED

VII B-2: Johnson, J. W. and R. L. Wiegel - Know Your Surf - Series 29, Issue 29, Institute of Engineering Research, University of California, Berkeley, California, September 1, 1950 (unpublished), RESTRICTED

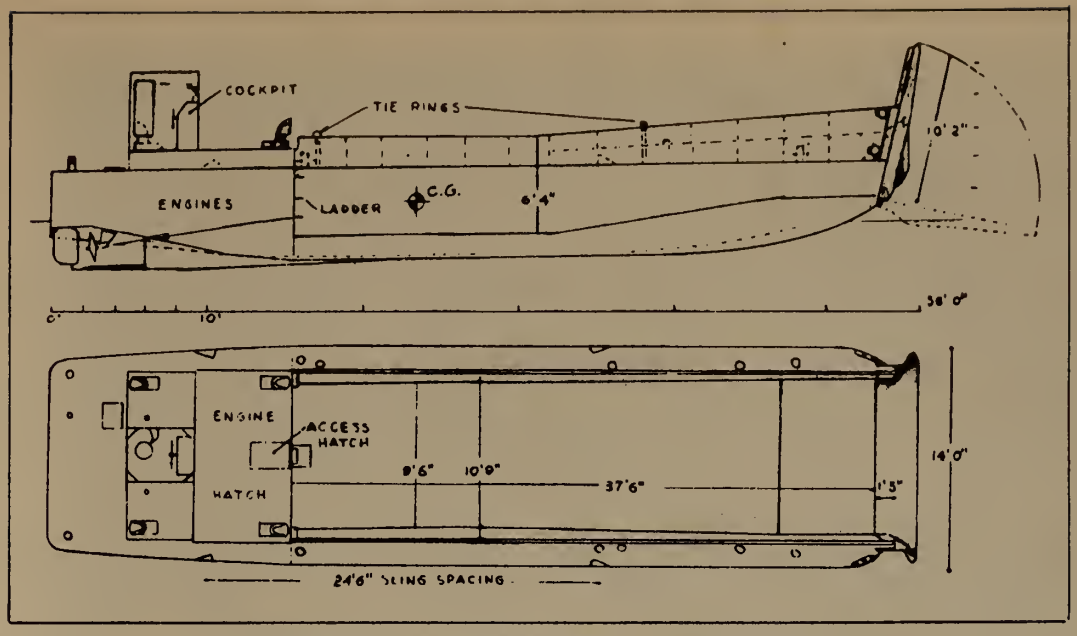
VII B-3: O'Brien, M. P. - Report on Surf Characteristics of Landing Craft - Series 3, Issue 104, Institute of Engineering Research, University of California, Berkeley, California, May 11, 1945, (unpublished), RESTRICTED

VII B-4: Scripps Institution of Oceanography - Effect of Surf and Beach Gradients on Landings with LCVPs and LCMs - Wave Project Report, Scripps Institution of Oceanography, La Jolla, California, (unpublished), RESTRICTED

VII B-5: Taylor, Lt. M. A., USMCR - Surf Conditions as Related to Landing Operations - Wave Project Report No. 22, Scripps Institution of Oceanography, La Jolla, California, October 12, 1944 (unpublished), RESTRICTED

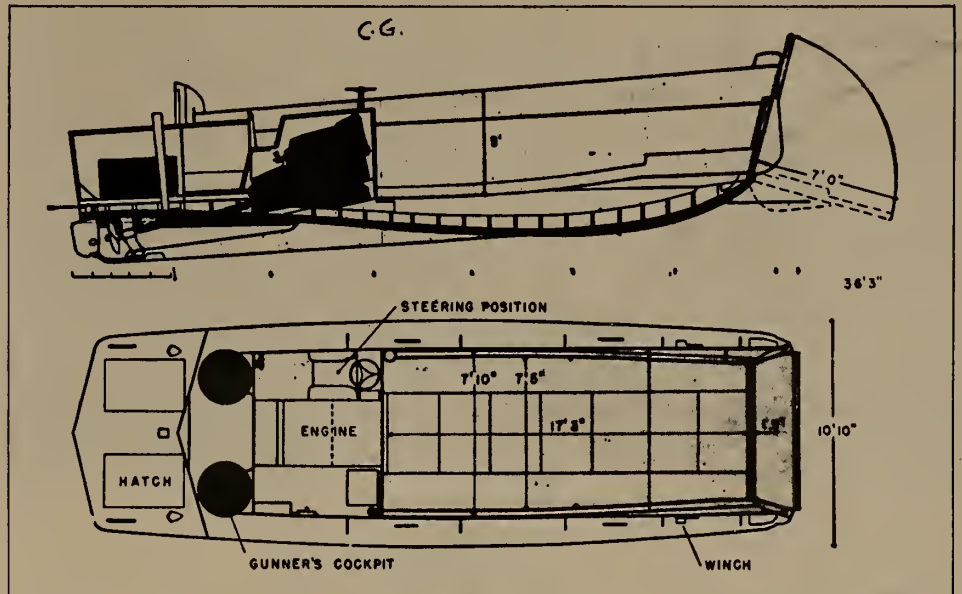
VII B-6: U. S. Navy - Surf Manual for Amphibious Operations - CominCh P-9, Headquarters of the Commander in Chief, October 8, 1945, RESTRICTED

VII B-7: Wiegel, R. L. and H. L. Kimberley, Lt. (USN) - Operation MIKI, Hawaiian Amphibious Phase (Ground Coverage) - Series 29, Issue 14, Institute of Engineering Research, University of California, Berkeley, California, October 25, 1949, (unpublished), RESTRICTED




a. ◀ **LANDING CRAFT, MECHANIZED (MARK 6)** **LCM(6)**

A Higgins type LCM(3) with 6' added to the hull amidships. Other characteristics are similar to LCM(3).




| | | | |
|---|---|---|---|
| Description | Essentially an improved LCV with steering control and gunners' cockpits in the hold. This design is standard equipment for all ships and supersedes LCV in production. | | |
| Capacity | 36 troops or 6,000-lb. vehicle or 8,100 lb. general cargo, provided center of gravity is kept low. Can be lowered when loaded only from davits. | | |
| Endurance | 102 miles. | Speed, 9 kts. | |
| Dimensions | Length 36' o. a. Beam, 10'5¼". | | |
| Displacement | Light, 18,000 lb. | Draft, 3' aft. 2'2" for'd. | |
| Armament | Two .30-cal. M. G. | Propulsion, 225-hp. Diesel. (Gray) or 250-hp. Gasoline (Hall-Scott) | |
| Armor | ¼" STS, ramp and sides. | | |
| Crew | 3. | | |

US

b. **LANDING CRAFT, VEHICLE, PERSONNEL** **LCVP**

FIGURE VII B-1

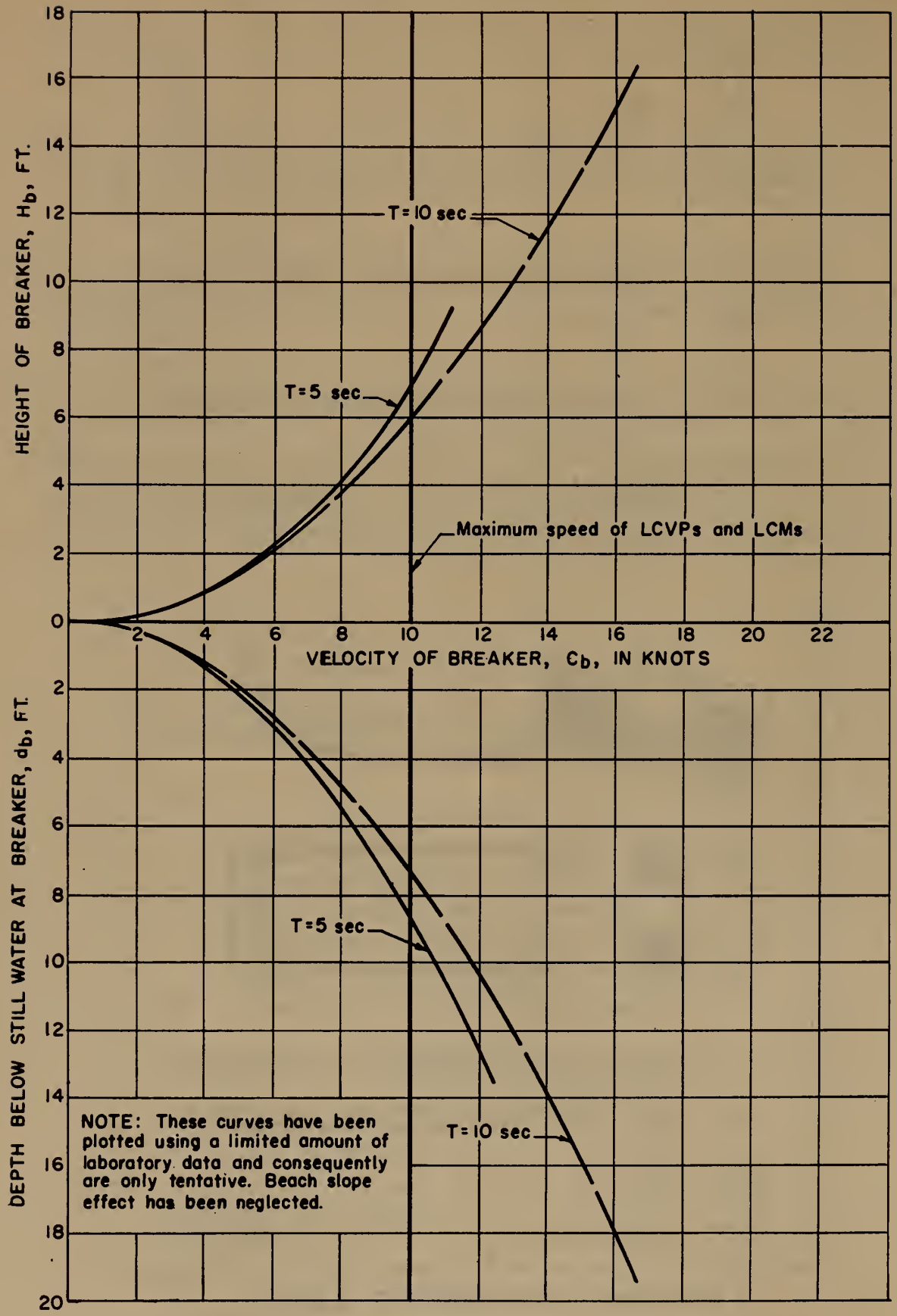



FIGURE VII B-2



FIGURE VII B-3 - Effect of surging breakers on landing craft. Long period waves on a steep beach.



a. Correct proceedure for retracting through the surf: backing out.



b. Incorrect proceedure: turning about in the surf.



c. LCVP broached in 3-foot surf.



Landing in a 2 to 3-foot surf.



c. Incorrect proceedure for retracting from the beach: turning about in the surf.



b. Personnel and signal units being discharged.

FIGURE VII B-5 - THE LCM

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FIGURE VIIB-6 - LCM(6) retracting properly, through a 12-foot plunging breaker; wet, but safe.

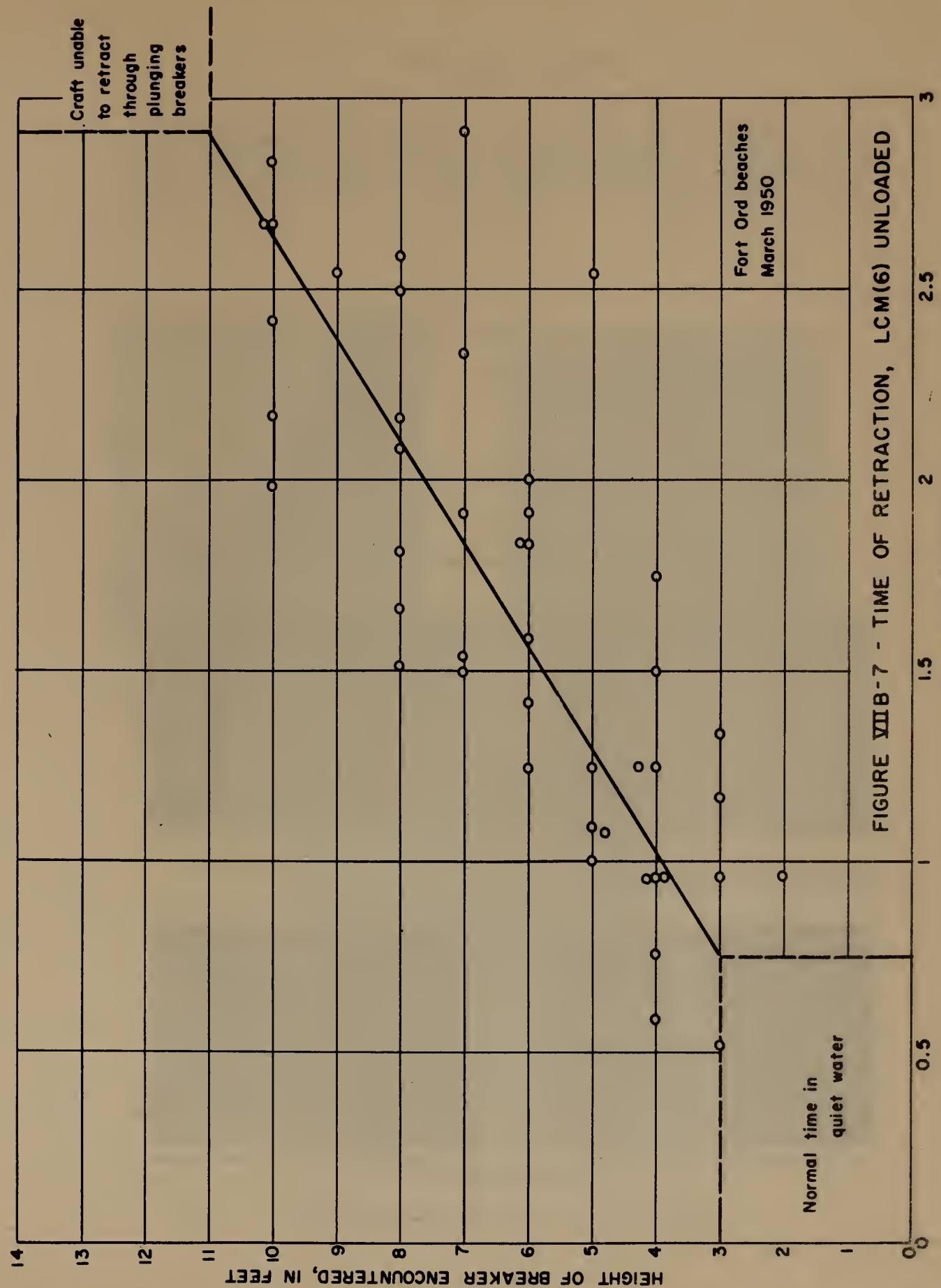
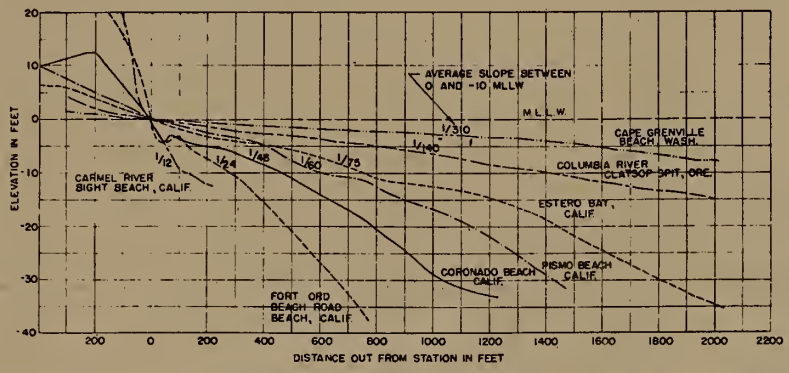



FIGURE VIIB-7 - TIME OF RETRACTION, LCM(6) UNLOADED

## FIGURE VIIB-8 - SOME BEACH AND SURF CONDITIONS

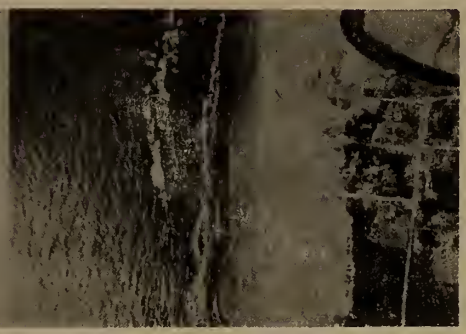
A plan form of a beach and its profile perpendicular to the shoreline are the long term results of the waves which act upon it, and of the supply and type of sand available. The beach slope is one of the predominating factors in determining the character of the surf. Beaches generally are classified as to slope, that is, flat, moderate, or steep. The following sketch shows the range of beach slopes that are found on the Pacific Coast of the United States. Aerial photographs showing typical beaches under heavy and light surf conditions indicate the wide variety of surf conditions that may exist on a particular beach depending upon the height of the offshore swell.




Typical Pacific Coast beaches showing the wide range of beach slopes that commonly are found



Heavy surf on a flat beach on the Washington Coast. Passing through this surf zone requires considerable time; hence it would be exhausting to the operator.

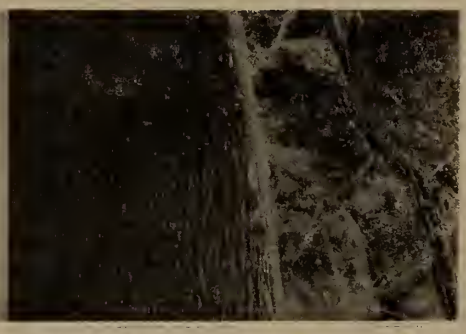


Light surf at the same locality as shown at the left. Compare widths of the surf zone.

SURF ON A FLAT BEACH



Heavy surf on a steep beach at Fort Ord, California.



Light surf at the same locality as shown at the left.

SURF ON A STEEP BEACH

FIGURE VIIB-9 - LITTORAL CURRENTS

A longshore current is set up within the surf zone when waves break at an angle with the shoreline. The currents flow parallel to the shoreline inside the breaker zone and are found most commonly along straight beaches. Their velocity increases with increasing breaker height, decreasing wave period, increasing angle of breaker-line with the beach, and increasing beach slope. The longshore current is much less than the velocity of the oscillating currents but it is relatively constant in direction with sustained velocities as high as three knots having been measured. Normally, velocities of from 1/4 to one knot can be expected. Strong longshore currents are effective in causing landing craft to broach.




Aeriel photograph of swell breaking at an angle to the shoreline, thus causing a longshore current in the direction shown.



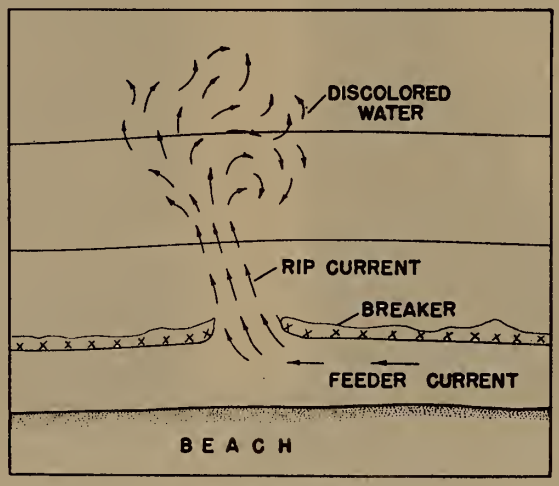
Photograph of landing craft broached as a result of the longshore current.



Some difficulty was encountered while attempting to drive jeeps from LCVPs which were careened by the surf, sometimes violently, endangering personnel. An attempt was made to steady and straighten out the craft by hawsers held by men on the beach. (Notice men standing between the jeep and side of LCVP.) November 25, 1946

FIGURE VIIB-10 - SURF RIPS

A type of current in the surf zone that may be of importance in certain areas is the rip current. Rip currents flow out from shore through the breaker line in narrow ripe and occur on almost all open coasts. They are caused by the waves piling water against the coast. This water flows along shore until bottom irregularities are such that it can escape seaward. As indicated by the following photograph, a rip current consists of three main parts: the feeder current, which flows parallel to the shore inside the breakers; the rip current proper, which flows through the breakers in a narrow band; and the seaward portion, where the current widens and slackens. Rip currents are easy to detect from the air but more difficult to detect from the ground. A ground observer usually can distinguish a rip by a stretch of unbroken water in the breaker line and patches of foam and discolored water offshore. Once rip currents have formed, they cut troughs in the sand and remain fairly constant in position until the wave conditions change. It has been found that, as rip currents cut channels through bars, it is usually best to operate LCM's and LCVP's in these ripe during low tides as then there is less chance of the craft becoming stuck. On the other hand, experience has also shown that it is best not to operate LVT's and Dukws in rip currents.




Idealized Rip Current
The Feeder Current may, and often does, approach from both sides.



Aerial Photograph showing Rip Current

# MANUAL OF AMPHIBIOUS OCEANOGRAPHY

## PART VII. SURF CHARACTERISTICS AND SURFMANSHIP

## C. AMPHIBIAN VEHICLES

by

LTjg. D. A. Ellis, CEC, USN, R. L. Wiegel and Lt. D. A. Patrick, CEC, USN

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## C. AMPHIBIAN VEHICLES

### 1. DUKW:*

a. Introduction:

The chief use of the Army DUKW** is to transfer cargo and personnel from ships offshore - through the surf zone - to dumps and storage points inland from the beach. DUKWs surpass such landing craft as the LCVP and LCM in two ways:

(i) DUKWs eliminate the extra cargo-transfer necessary with LCVPs, LCMs etc. (at the beach).

(ii) DUKWs can navigate surf zones impossible for these landing craft.

The purpose of this section is to describe the beach and surf conditions affecting DUKW operation. The type of surf and of beach are interrelated in a complex way, and each variation creates special problems for DUKW operation. (For example, with a surf of a given height, a DUKW can get safely through to a beach of one sort, but have great difficulty landing on a different type beach.) Figure VII C-2 shows several scenes of DUKWs at work in an amphibious operation.

The plan of this report:

First: Discussion of some surf factors affecting DUKW operation, including breaker height, longshore currents, and wave variability.

Second: Discussion of some beach factors affecting DUKW operation, and combinations of beach and surf conditions affecting DUKW operation.

Third: Final remarks on DUKW operation in the surf.

Finally, photographic illustrations are appended.

b. Surf Factors Affecting DUKW Operation:

There are four surf factors particularly important to DUKW operation:

(i) Type of breaker (whether they are plunging, spilling or surging),

(ii) Angle of breakers in relation to the shoreline,

(iii) Period of breakers, and

(iv) Wave variability.

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* Note: Most of this information has been taken from Reference VII C-2.

** Note: The DUKW is officially defined as an Amphibian Truck, 2½ tons, 6x6. (See ONI/226, 1944, VII C-3)
Figure VII C-1 shows the relative dimensions of Army DUKWs.
As previous reports have described their operation, use and maintenance these details are not repeated. See Reference VII C-1.

(1) Types of breakers: In water seaward of the surf, DUKWs behave much like other small, stable craft. They have often operated without difficulty in waves 12 feet high. In landing, the most important consideration is the mode with which waves break. Waves "peak up" after "feeling the bottom", and keep getting higher until they break. Breakers are of three kinds: Plunging, spilling and surging; however, they shade from one type to another.

(a) Plunging breakers: These are the most violent, and hit a craft or vehicle with the greatest force. The wave peaks up until its foreward face is a nearly vertical wall of water, then its crest curls over and descends violently into the preceding trough - where the surface is essentially horizontal. (Figure VII C-3). The breaking is almost instantaneous and is usually limited to a narrow zone.

The experienced DUKW driver can sometimes avoid meeting a breaker entirely by moving in the comparative lull between breakers, but this requires perfect timing. Otherwise, the DUKW may meet the plunging wave at its most destructive stage.

For routine work, DUKWs should not be used through plunging breakers higher than 8 feet. For emergencies, however, they can be operated by skilled drivers in plunging breakers as high as 16 feet. Extreme caution is necessary to reduce hazards. (For example, in breakers as high as this the windshield is often shattered.) (Figures VII C-4 and VII C-5).

(b) Spilling breakers: These break more gradually and over a greater distance than plunging breakers. The wave peaks up until it is very steep. Then only the top of the crest curls over and descends - not into the preceding trough like a plunging breaker - but rather on the forward slope of the advancing wave, where it then slides down into the trough (Figure VII C-3).

A spilling wave starts to break at a number of different points, and this breaking spreads longitudinally along the crest. Thus at any one time, some parts of the wave are turbulent foam and others are unbroken crest. As these breaking sections merge into each other, the wave at the junction point invariably plunges.

DUKW drivers tend to steer their course to the unbroken portions of spilling breakers. This is fine if that area can be crossed before the wave breaks. It is hazardous, however, and the driver must frequently cross at the breaking or broken part (Figure VII C-6).

For routine operations, and light loads DUKWs should not be used through a surf of spilling breakers more than 12 feet high. However, DUKWs driven by experienced personnel can with great care traverse spilling breakers up to 16 to 20 feet in height. This is dangerous, and should be done only in emergencies.

(c) Surging breakers: These occur only where there exists the combination of long period waves breaking on steep beaches. These breakers, instead of using up part of their force by breaking normally (i.e. plunging or spilling), surge high up the beach face. Surging breakers are usually long period waves (15 seconds or so) and approach the beach in a long line. Due to the long period of breaking, and the single line of the breakers, the DUKW driver should be able to pass through ten foot surging breakers without great difficulty. The upper limit for skilled operators in emergencies has not yet been determined.

Because surging breakers only occur on steep beaches, DUKWs are liable to broach when trying to land as they cannot always climb the steep beach face and get away from the succeeding breakers. Thus the DUKW may become swamped (as occured at Iwo Jima).

(2) Angle of Breakers: (Figure VII C-7). Another surf factor important to amphibious operations is the angle between the breakers and the beach. Breakers approaching the shoreline at an angle set up "littoral" (or "longshore") currents in the surf zone which are parallel to the shore. High waves of great angle may set up a littoral drift that moves as fast as three or four knots.

Waves breaking at an angle with the beach make any landing operation more hazardous. In addition to the difficulty of getting through the breakers, there is the added obstacle of the littoral current tendency to cause broaching on the beach face. DUKWs are not critically affected once they reach land - providing the beach is not so steep that they can't move up it.

To make a safe landing (through waves breaking at an angle to the shore), the DUKW driver should first estimate the direction and total distance of the littoral drift, and then steer his course to meet the breakers head-on or directly astern. Where there is a bar, the highest currents are usually in the channel which lies between the bar and the beach. If there is no bar, there are currents throughout the surf zone, but their intensities vary greatly at different parts.

(3) Period of Breakers: Another surf factor affecting amphibious operations is the period of breakers. In a wide surf zone with storm waves (of from 6 to 12 seconds), there will be a large number of breakers. These will be tiresome to the DUKW driver, and may also make him lose his bearing. On the other hand, on a steep beach with long period waves (12 to 20 seconds) the DUKW may be able to pass through the breaker zone between waves.

(4) Wave Variability: (Figure VII C-8 and 9). Waves are variable along the beach and with time at any given point off the beach. Both of these phenomena should be taken advantage of. Land at the point of lowest breakers, if practicable. Land during lulls in the surf, if practicable. If the surf is low, of course, it is usually not necessary to worry much about such procedures. However, in high surf it may well be the only manner in which supplies may be brought ashore.

Before entering a harbor of breaking waves, fisherman commonly spend an hour or so counting the number of breakers and timing the lulls. They can then time their approach during favorable periods of low breakers. DUKW drivers should follow this same practice in planning any hazardous landing. Especially when beaches are steep, it is most important to judge the variability of the surf and traverse it at the most favorable time.

c. Beach Factors Affecting DUKW Operation:

Beach factors of importance to DUKW operation can be classified under five headings:

(i) Steep Beaches - with slopes steeper than 1:15

(ii) Moderate Beaches - with slopes of 1:15 - 1:30

(iii) Gentle Beaches - with slopes of 1:30 - 1:60

(iv) Mild Beaches - with slopes of 1:60 - 1:120

(v) Flat Beaches - with slopes less than 1:120

They may also be classified according to the combination of beach and surf;

(i) Effect of exposure on beach type

(ii) The seasonal change of beach type

(1) Steep Beaches:

(a) Definition: Beaches having slopes steeper than 1:15.

(b) Wave pattern: On steep beaches there is normally only one wave in the surf zone at any one time.

(c) Beach composition: Usually the beach is composed of coarse sand particles or gravel.

(d) Beach profile: Changes rapidly during different sea conditions. It steepens during calm seas, tends to flatten during a period of high storm waves.

(e) DUKW operation on steep beaches: In landing on a steep beach, the first criteria is to make sure that the DUKW can climb from the surf zone onto the steep beach. If so, the driver may be able to wait beyond the breakers for a lull, and then cross the surf zone without meeting any breaking (or maximum) waves. In approaching the beach, the DUKW wheels must be engaged so that no time is lost at the critical transition on the beach face.

In retracting from the beach, the principal hazard is that the DUKWs bow becomes afloat before its stern and rudder are in the water. This leaves the driver without means to direct his course, and his DUKW may be broached or capsized by a slight current or wave at an angle. It is therefore important - once the trip is started - to keep going full speed in second gear, low range. It is also necessary to engage the screw. While this may bend the propeller shaft, it is unsafe to enter the breaker zone if the breakers are large, without first doing this. (Figure VII C-9).

(2) Moderate Beaches:

(a) Definition: Beaches having slopes of 1:15 to 1:30.

(b) Beach profile: Unless they are in partly protected waters, moderate beaches usually have one offshore bar. (Rarely do they have more than one.) When there are high storm waves, this bar becomes more pronounced, while the beach face becomes flatter. During a period of low waves, the bar tends to disappear or become discontinuous, while the beach face becomes steeper.

(c) Wave pattern: Plunging breakers are less common, spilling breakers more frequent. If there is a bar, there may be plunging breakers at low tide, which become spilling at high tide. High, long period swell will usually plunge at both high and low tide.

When there is a bar, currents are always present in the channel which lies between the bar and the beach. When incoming waves are parallel to the beach, these currents are of low velocity, but when the breakers approach at an angle, these currents are often one to two knots and under severe conditions may reach a velocity of four knots.

These currents normally follow the channel for some distance, and then flow seaward, cutting a channel through the bar at various low points 400 to 5,000 feet apart. These currents are called "surf rips" and often have high velocities.

Where the beach face is steep, strong currents may also exist in the inner surf zone, but where the beach face is moderate, they will be weak.

(d) DUKW operation on moderate beaches: Sometimes all the hazards of steep beaches may be encountered. However, the hazards are usually temporary and are limited to only certain parts of the beach. They can thus be avoided.

(3) Gentle Beaches:

(a) Definition: Beaches having slopes of from 1:30 to 1:60.

(b) Composition: Medium size sand.

(c) Beach profile: Has one to two offshore bars except during long periods of little wave action.

(d) Wave pattern: Breakers are usually the spilling type. Often high waves which break on an outer bar reform and break again on an inner bar; however, they usually loose so much energy during the first break that they are not as hazardous on inner bar. However, it may be possible that the breaker on the outer bar may be very high, but spilling, while the reformed wave may plunge on the inner bar and although lower be more hazardous.

(e) DUKW operation: For the same height breakers as on steep beaches, this type of beach is less difficult as the waves usually spill. The surf zones are not as wide as on the Mild or Flat beaches and so the drivers fatigue is not so serious a problem.

(4) Mild and Flat Beaches:*

(a) Definition: Beaches having slopes less than 1:60.

(b) Composition: Mild and flat beaches are usually composed of fine sand, but a pea gravel or gravel beach face is occasionally found at the mouths of small creeks.

(c) Beach profile: Unless partly protected, beaches of mild and flat gradients will usually have several offshore bars at all seasons. The beach faces are always suitable for amphibious operations except at extreme high tides during heavy weather - when the beach may be impassable due to eroding at certain points.

(d) Wave pattern: Plunging breakers are less common, and spilling breakers are the general rule. When there is a combination of long period swell

* In regard to DUKW operations, mild and flat beaches may be considered together.

and short period wind waves, a certain amount of spilling will take place on the outer bar. This spilling may obliterate the wind waves, and the dominant swell may reform in the channel behind the bar and plunge on one of the subsequent bars or upon the beach face. It has, of course, lost energy in the spilling process, and will not be as high as if it had plunged previously. Even though the spilling surf is quite moderate on the outer bars, this plunging of breakers on one of the inner bars may make the surf impassable.

The currents may be very strong in the channels between the bars, especially when waves are breaking at an angle. There is very little current at the beach face under any circumstances. The flow of water which is built up between the bars escapes at certain low points or partial channels between the bars. The channels throughout the different bars may not be continuous, in fact, where a channel occurs through the inner bar the next bar to seaward is likely to be more shoal at this point than elsewhere.

(e) DUKW Operation: These types of beach faces are never too steep for DUKW operation, but are occasionally impassable at high tide due to erosion taking place into the backshore.

One of the principal hazards of such a beach results from the inability of an operator to judge the height of the breakers and to estimate the intensity of the waves which are arriving offshore. A 20 foot breaker plunging at a distance of a half mile offshore will not appear hazardous from the beach face, and several sorties have been attempted under conditions which would have never been considered had the breakers been at a closer range. The great length of time required for the passage through this type of surf imposes a severe strain upon the craft and upon the personnel. (Figures VII C-10, 11 and 12). Drivers have frequently spend three quarters of an hour bucking out through surf of this nature at full power and almost constantly subjected to strong motion and impact. This can frequently lead to confusion and loss of orientation. It is routine to lose the wind-shield wipers, shatter the windshield and tear the forward canvas (Figure VII C-5b). This makes the already poor visibility worse. A driver must be experienced enough to keep the DUKW head-on into the breakers in spite of this difficulty. Advantage can be taken of the comparatively calm water existing in the channels between bars. A driver can regain his bearings and estimate the intensity of surf arriving upon the next bar, or turn the craft around if the maneuver is carried out carefully.

(5) The Effect of Exposure on Beach Type: In general, beaches subjected to waves coming from a variety of directions and having a variety of periods and heights are of a much more complex nature than beaches affected by more limited conditions.

(i) The simplest type beach is one in the head of a bay or bight where the only waves reaching the beach are those which have come through the entrance and have refracted and diffracted. This beach is affected by only a few type waves because many different periods of waves have been screened out. The waves arrive approximately parallel to this type beach, unless disturbed by freshets or tidal currents. The waves, if originally at an angle to the beach, produce a transport of the beach material in such a direction and manner so as to bring about this balanced condition. After this balance has been obtained, the waves striking the beach parallel cause a general rise in water level within the surf zone. The variation of height

of any breaker throughout the line of the beach is gradual and there is no direct compulsion for this increased head of water to escape in any particular manner. Thus small irregularities in the beach set up a series of partial channels throughout the surf where this transported water escapes.

On beaches where conditions are nearly constant, these cusp-like channels are usually fairly evenly spaced for a particular beach; they may vary in length (along the beach) from 20 to 500 feet or more depending on the beach. These cusp ridges are composed of "soft" sand deposited by wave action and interfere with the activities of DUKWs both in landing and travelling along the beach.

A landing point always should be selected so that the DUKW lands in the trough of these cusps rather than upon the crest, because the sand is more firm and there won't be as much chance of the DUKW bogging down while trying to leave the water.

(ii) Beaches in large wide open bays, or extremely long beaches which are subjected to waves from a constant angle, frequently do not set up this balanced condition mentioned above. This is because the points or headlands confining the beach are so limited in seaward extent, that regardless of the transport of materials in the direction in which the waves are travelling, the transport merely continues indefinitely without effectively changing the trend of the beach. Such beaches have relatively simple profiles since forces acting upon them are of a nearly constant direction.

(iii) Beaches usually have a complex profile when affected by waves coming from many directions or when the waves are of varying types. They are affected by currents which change in direction from day to day and their profile differs greatly at closely adjacent points. They have irregular, discontinuous bars of varying number; flat and steep sections; and partial channels or rips throughout the surf zone at irregular intervals of 300 to 5,000 feet. There are several examples of this sort of beach along the Pacific coast, where a continuous beach curves farther and farther to seaward at the southern extremity. In general, if any part of the beach has some protection which eliminates waves from one direction, this will produce a more regular profile and beach type than elsewhere.

(6) The Seasonal Change of Beach Type (Figure VII C-13): During extended periods of low waves, beaches may build high berms on the foreshore with very steep profiles of the beach face. Bars and underwater berms tend to disappear or become discontinuous. During periods of high waves, the beach face becomes less steep and the bars or underwater berms become more pronounced. Of course this flattening of the beach face involves cutting back of the berm, and temporary steep banks may be formed which seriously impede the operation of an amphibious vehicle. Waves arriving parallel to the beach over long periods form cusps. When the wave conditions change, these cusps will be truncated or differently spaced. Where the crest of each cusp has been cut away, a series of steep escarpments develops, and soft sand is deposited in the troughs of the old cusps. If the cusps were originally large, this may produce difficult operating conditions for the DUKW. It may be difficult to operate on any beach which is undergoing, or has recently undergone, a distinct change.

d. Final Remarks on DUKW Operation in the Surf:

(i) The inherent variability in wave heights at any time is a favorable factor for the operation of a DUKW.

(ii) It is extremely important to judge the breaker height. In general, at any point on a particular beach where the breakers are occurring farthest from shore, the breakers will be highest. This is due to the fact that there must be a bottom configuration offshore from such a point - tantamount to a submarine ridge. Such configuration makes the waves converge upon the portion of the beach immediately shoreward of itself. Conversely, surf regions where the outer breakers are constantly breaking closer to shore will generally be regions of lower breakers. This is due to the fact that here a condition of divergence over a small channel must exist.

(iii) When a DUKW is in breakers, or in water that is shallow enough for the wheels to touch, it is extremely dangerous for the vehicle to be other than at right angles to the waves. Broaching and capsizing are the imminent dangers in heavy surf whenever the course taken is other than normal to the breaker crest.

(iv) The DUKW under full power can be made to surfboard on breaking waves. This is often an advantage when approaching shore, as the surf zone can be traversed faster than would otherwise be possible. The DUKW can maintain full speed ahead as long as its position is not hazardous. It is apparently more apt to surfboard on long period waves than on short period waves. On long period waves also, its position on the crest is farther to seaward. When the DUKW surfboards on short period waves, it is most likely to take a position ahead of the crest, with its bow in the trough and its stern riding on the advancing slope. This seems the more dangerous position, as any slight deviation from a normal course would result in heavy turning and sudden broaching. However, the DUKW shows less tendency to broach than do LCVPs and LCMs. These craft have been observed to broach very rapidly and violently under these circumstances.

(v) It is inadvisable to back down to avoid surfboarding. The surfboarding can usually be terminated at any time by decreasing motor speed.

(vi) If a DUKW starts surfboarding on a plunging breaker before it reaches the breaking point, the wake and bow waves produced by the DUKW will modify the breaker in the region around the DUKW. The wave then spills in this area. This is of course preferable to having a plunging breaker overtake the DUKW at the plunge point. The DUKW should only be slowed down to less than full speed in two cases: (It has been found advisable to proceed at full speed at all other times when in the surf.)

When going seaward through a surf zone where the breakers are consistently plunging in a restricted zone, the DUKW may be slowed down to avoid meeting a breaker as it plunges (if there is a question as to where the wave will break, it is better to meet it with sufficient steerage way than to have it strike the DUKW without way on).

When travelling shoreward, the engine speed may be decreased to avoid or terminate surfboarding if the DUKW is riding the wave well up on the crest. However, if the wave is completely astern, this cannot be done safely.

(vii) Overloading of the DUKW is to be avoided. The maximum safe payload in moderate surf is 5,000 pounds. In heavier seas, a lighter load is preferable. During the heat of battle and the necessity for moving vast quantities of supplies ashore in a limited time there is a tendency to overload craft and lighters. Case histories of amphibious operations frequently show losses due to overloading of boats or vehicles. Overloading DUKWs result in:

Decreased freeboard, which allows breakers to swamp the craft.

On steep beaches, the DUKW may be unable to traverse the beach slope and is subjected to full effect of the breakers.

A heavily loaded DUKW, unable to negotiate the surf and beach, is hampered in trying to re-embark safely onto LSTs. In moderate seas it will be unable to climb the ramp of the LST and must seek protection elsewhere.

(viii) There have been cases of DUKWs being sunk in the surf as a result of water getting into the engine compartment (and wetting the electrical system) because the engine hatch covers were not dogged down.

(ix) A DUKW operating in quiet water (harbors, bays, lakes, rivers) is not subjected to violent wave forces. The greatest difficulty in such areas is finding a suitable site for crossing the shoreline adjacent to the waters edge as they are often composed of muds, (Figure VII C-14).

(x) DUKWs must be manned by trained drivers when operating in the surf. Otherwise, a high mortality rate will result from lack of proper maintenance and mishandling in the surf and along-side ships.

## 2. Landing Vehicles, Tracked (LVTs):*

### a. Introduction:

This section is based on observation and analysis of several hundred LVT sorties through the surf on various beaches along the Pacific Coasts of the United States and Hawaii. Results are included from special tests made by



* Most of the information in this section was obtained from references VII C-4, 6 and 8.

personnel of a Naval Research Project, jointly conducted by the University of California and the U. S. Marine Corps. These sorties, together with model studies simulating field conditions revealed certain relationships between:

(i) Breaker dimensions

(ii) Beach and ocean bottom configurations

(iii) Vehicle position and behavior

It is the purpose of this section to discuss these relationships.

The amphibian tractors used by the U. S. Marine Corps as of the date of this manual (September, 1951) are the LVT(A)(5) and the LVT(3)C. (The latter was formerly the model LVT(3), but it now has a cargo compartment cover and certain other modifications.) Since 1941 the whole LVT program of the Marine Corps and the Bureau of Ships has been changing. Briefly, current plans for LVTs are twofold: to modernize and rehabilitate present models, and to manufacture new models. Although some of the data presented here will not necessarily apply to the new models, most may be applied by proper interpretation with modeltests - possibly without the need for extensive field tests.

Because many of the illustrations in this section have been taken from a report of special field tests (reference VII C-8), differences between the test vehicles and the production models should be explained. Briefly, the test vehicles were painted in alternate one-foot black and white bands and fitted with corner poles for photogrammetric purposes; were fitted to accomodate removable pitch recorder apparatus; and one LVT(3)C was fitted with a remote control apparatus which could operate the vehicle from a shore station when tests were desired in surf too dangerous for personnel. These minor changes are illustrated in Figure VII C-16. The principle characteristics of the production models are presented in Table VII C-I.

TABLE VII C-I - PRINCIPLE CHARACTERISTICS OF LVT(3)C and LVT(A)(5)

| Characteristics | LVT(3)C* | LVT(A)(5) |
|---|---|---|
| Length, overall | 24.92 feet | 26.1 feet ** |
| Beam, overall | 11.20 feet | 10.8 feet ** |
| Height, overall | 8.4 feet | 8.2 feet ** |
| Weight, Unloaded | 38,775 pounds | 36,000 pounds ** |
| Weight, Loaded | Capacity not reported | 38,000 pounds ** |
| Light Draft above hull bottom | 2.92 feet | 2.7 feet *** |
| Appraoch Angle | 33° (estimated) | |
| Departure Angle | 30° | |
| Hull bottom to center of buoyancy | 1.61 feet | 1.7 feet *** |
| Hull bottom to center of gravity | 3.04 feet | |
| ℄ Drive sprocket to center of gravity | 9.82 feet | |
| Metacentric Height, Longitudinal | 15.29 feet | |
| Natural Period of Pitch | 2.75 seconds | |
| Radius of Gyration | 9.73 feet | |

* Data from information supplied by San Francisco Naval Shipyard, August, 1950.

** Data from: The LVT(3) and the LVT(A)(5) Training, Operations, and Employment, (Tentative draft), U. S. Marine Corps, Landing Vehicle Tracked, Manual Board, Camp Joseph H. Pendleton, 1949, p. 109. (RESTRICTED)

*** Data from: BuShips Drawing "LVT II Displacement and Other Curves", No. 552105 of March 17, 1944, approved for LVT(A)(5), July 16, 1945.

b. Relationships between Oceanographic Factors and Operations of LVTS:

(1) Introduction:

Certain relationships exist between oceanographic factors and operations of LVTs as shown by field and laboratory investigations. The following five factors most vitally affect tractor operation:

(i) breaker characteristics (type, height, period and velocity),

(ii) position of the craft relative to the wave at the time of breaking,

(iii) craft velocity,

(iv) slope of the beach, and

(v) currents, (surf rips and longshore).

Before discussing the effects of these various factors on vehicle operation, it is necessary to understand what a surf casualty is, and how it could be caused. A surf casualty is any set of natural occurences between the breaker line and shore which prevents a task unit (an LVT) from carrying out its assigned mission on schedule. Effects of enemy action have not been considered. The surf zone, and more specifically the breaker zone, has been found to be the most critical area for an LVT travelling from ship to shore and from shore to ship. An LVT may become a surf casualty, in this area by:

(i) pitching or rolling over,

(ii) swamping from wave inundation,

(iii) broaching (primarily caused by a mechanical failure with resulting loss of control), and

(iv) sinking from loss of hull integrity (split seams, punctures, etc.).

The pitching or rolling over has been found to cause the most serious casualties because personnel and cargo are oftentrapped inside an inverted craft in water too deep for immediate salvage. Swamping was found not to be a cause of casualties for the present models, since both are covered effectively enough to prevent large quantities of water from coming aboard. Although - on a few occasions sea water entered the LVT(3)C test vehicles through one of the engine air intake louvres, and caused the engine to sputter for a short time, this trouble has been corrected on subsequent LVT(3)Cs. One case of broaching has been reported with the LVT(3)C, though the vehicle recovered in time to proceed safely to shore. Sinking from loss of hull integrity did not prove to be of any concern during these studies, as the LVTs were all on sand beaches (except for one cobble slope): it would, however, be a real danger when operating over rocky or coral reefs.

(2) Operational Surf Limits:

Critical breaker characteristics are particularly difficult to evaluate from field tests since the four main characteristics (type, height, period and velocity) may exist in so many different combinations. Waves have been classified as spilling, plunging and surging. As ocean waves are irregular and variable, only generalizations can be made. Fortunately, delicate refinements are not necessary to

establish many valid conclusions.

For LVTs, the most dangerous type of breaker is the plunging type. Limited observations of amphibious tractors in surging breakers, indicate that these are probably the least dangerous breakers. Spilling breakers seem to lie in between these two limits. Other factors being equal, an LVT can land safely through a spilling breaker about 50% higher than a plunging breaker.

During field tests, LVTs passed safely through the surf zone in what operating personnel believed to be the upper limits of breaker heights. In two cases, one with the LVT(3)C in an 8 foot plunging breaker, and the other with the LVT(A)(5) in a 12 foot spilling breaker, paint was scraped from the bows when they grounded at the highest pitch. One of these critical landings of the LVT(3)C is shown in matched simultaneous photographs from two camera stations ashore (Figure VII C-17). The motion picture sequence of this same landing is shown in Figure VII C-18. Superficial damage done to both tractors is shown in Figure VII C-19 (both were nearly turned over).

Experienced tractor personnel consider the following to be the limiting heights of breakers for operation of current model LVTs by average drivers:

8 foot high plunging breakers

12 foot high spilling breakers

6-7 foot were the highest surging breakers through which an LVT landed, (Figure VII C-20). No difficulty was encountered. The tractor did not pitch much and was pushed onto the beach face. Being a track-lying vehicle, it immediately took hold and continued up the beach face onto dry ground. Laboratory tests of LVTs in surging breakers with heights corresponding to prototype heights up to 9 feet have indicated that they would have no trouble. Caution should be observed, however, as this type of wave can easily plunge rather than surge if a slight change occurs in the deep-water wave characteristics.

These are the limits approaching shore. Observations definitely show that, where swamping is not a problem, the LVT(3)C and (A)(5) amphibian tractors can retract through a higher surf than they can land through. Figures VII C-21, 22 and 23 show some sequence photographs of LVTs retracting and landing. Figure VII C-24 shows a comparison of the pitch record for landing and retracting in a similar surf. However the tendency to swamp while retracting is worse for the uncovered type LVTs. For example, two LVT(3)s, (without cargo compartment covers) were swamped while going out through the surf off Aliso Canyon, Camp Pendleton, Oceanside, California. The sinkings took place about an hour and a half apart, and both seemed to be caused primarily by an ignition failure after sea water entered the cargo compartment and got into the engine compartments. No personnel were lost. Examination of wave records showed that, as usual, the surf consisted of a series of high and low breakers. Evidently, the vehicles were hit consecutively by a series of high breakers and were thus swamped. The significant breaker height was approximately 6 feet, with the maximum breakers being about 10 feet high (14 second period). With the present model LVTs, however, very little water comes aboard, and of insufficient quantity to cause concern.

(3) Effect of Pitch on Firing of Howitzer from an Armored LVT:

The forward pitching of the LVT is the more important in respect to stability. But the reverse pitching may be important when firing howitzers and other heavier weapons from an armored LVT at sea. When the vehicle is subjected to a reverse pitch, the angle of depression of the weapon may be limited. Shallow-water waves are less steep on the seaward side than on the beach side, and this steepness increases gradually until the wave breaks. In a series of pitch records of 11 landings of the LVT(A)(5), the maximum recorded reverse pitch was about 15 degrees when the craft was just seaward of a wave as it broke. The average reverse pitch outside the breakers was about 3 or 4 degrees. From a study of the dynamics of a vehicle subjected to wave action, it is apparent that it is in the most stable position when in the trough of a wave.

(4) Effect of Breaker Velocity: (Reference VII C-5)

Two types of velocities are associated with a breaker. One is the orbital velocity of water particles within the waves; the other is the velocity of the wave moving towards shore (the "wave velocity").

The wave velocity, relative to the vehicle velocity, is important for two reasons: The first reason is that the greater the breaker velocity, the greater the number of breakers that the vehicles will encounter. If the breaker velocity were less than the vehicle velocity, the vehicles could follow a breaker in and hence never be subjected to its force. Figure VII C-8 shows a graph of breaker velocity as a function of breaker height and depth of breaking (on the basis of some limited laboratory data). All but the lowest breakers moved ahead with velocities greater than an LVT. The second reason is the relative time that an LVT is subject to the forces of a breaking wave. Some laboratory experiments indicate that the vehicle, if dead in the surf, would be pitched to a smaller angle than if it were moving shoreward (Reference VII C-6). On the other hand, if the vehicle has any appreciable shoreward velocity, the maximum angle to which it would be pitched appeared to stay constant. However, from a control standpoint, it seems better to have the vehicle under full headway.

The other type of velocity is that of the water particles in the waves. Figure VII C-26 shows three sketches of the velocities of these particles. These sketches were prepared from motion pictures of waves in a glass-walled channel. (Reference VII C-7) A landing vehicle advancing shoreward through the surf has its stern lifted by the forward face of the crest (Figure VII C-27). As the stern is lifted, the bow sinks deeper in the preceding trough. In this position, the stern is being forced toward the beach by the water in the crest, while the bow is being forced seaward by the back flow. These two forces, acting in opposite directions, tend to turn the vehicle over, or if at an angle to the breaker, broach it. The distance between the maximum opposing forces is approximately twice the height of the wave; hence, an amphibian tractor in a 10 to 12 foot breaker would have the maximum effective turning couple acting upon it. Of course, as the breaker increases in height, the forces increase and although the maximum forces are not acting as a turning couple, the truning couple probably still increases.

It can be seen that if the center of the vehicle is at the crest of the breaker, or just behind it, the net force tends to push the vehicle forward and so it "surf-boards" shoreward. The velocity distribution in the trough preceding the breaker is such that the net force pushes the vehicle seaward. This retards the forward motion of the vehicle. In fact, during several runs into shore through the surf, and LVT was actually

pushed seaward even though it was under full speed ahead. As a result, the vehicle was swept back into the crest of the following breaker, and was thus subjected to the full force of the breaking wave.

(5) Relative Position of Amphibian Vehicles:

The position of the LVT relative to the wave at the time of breaking is probably the most important single factor affecting the vehicle's safety. Figure VII C-27 shows the effect of vehicle position upon pitching, for otherwise identical conditions. These sketches were drawn from motion pictures taken of a LVT(A)(5) model in a wave channel and were verified by actual field tests. Figure VII C-28 shows this effect graphically in more detail. Obviously, the effect of position is very critical. If the vehicle were at the crest of the wave as it broke, or just seaward of it, it would be surf-boarded shoreward. If it were about one-eighth of its length ahead of the crest it would experience the maximum pitch. If it were a vehicle length ahead of the crest at the time of breaking, it would experience only a relatively small pitch when the broken wave hit it.

As an example of how this relative vehicle position lessens the chance for surf accidents, consider a breaker with a length of 200 feet at the place of breaking. There is just one chance in ten that the vehicle would be in the critical position, relative to the breaker, when hit. This is a conservative figure. Another way of saying this is that even when a vehicle encounters a wave, there is only a small chance that it will encounter the wave immediately after it breaks.

(6) Effect of Beach Profile:

The slope of a beach will be considered in two sections: the immediate offshore section and the beach face. The effects (on LVT operation) of the profile of the offshore section of the beach are twofold. One is the effect of offshore bars and the other is the effect of the slope. As LVTs are track-laying vehicles, offshore bars do not present as serious an obstacle as they do to LCVP, LCM and landing ships. At low tide the bar, or bars, may not be sufficiently covered with water for LVTs to freely float over them. However, these vehicles can immediately run across the bars if both tracks ground squarely. (Effect of only one-track grounding is discussed below.) This same advantage was discovered in crossing coral reefs during the Pacific campaigns of World War II.

The offshore slope is important in vehicle operation primarily for two reasons. One is the effect of slope on breaker characteristics. The other is the effect of slope on the number of breakers that a vehicle encounters. For identical deep-water waves, the flatter the beach - the wider the surf zone - and hence, the greater the number of breakers that a vehicle encounters. Figure VII C-29 and VII C-30 show aerial photographs (identical scales) of a steep beach and a flat beach. Since the passage through a line of breakers is the crisis for amphibian tractors, several lines of breakers mean several crises, so that some difficulties might be cumulative, producing an unsafe condition. In addition, in a very wide surf zone, driver fatigue becomes important as does also the problem of maintaining proper course.

Observations made in both the field and in the laboratory show that, in general, the depth of water under the breaker is greater on the flatter beaches. (Reference VII C-5) Analysis of previous LVT(3) casualties indicate that many are caused when the vehicle pitches to a maximum, its bow hits bottom: then the vehicle either turns over, or pivots on one corner and swings about towards sea and fills with water. The latter alternative is now minimized because of the

new covers. On flatter beaches, because the water under the breaker is deeper for a given wave, the chance that the LVT will right itself before grounding is better. The offshore section of the beach, if irregular with small steep bars, may affect LVT operation in another singular way. If the LVT grounds on the bar, but not squarely with both tracks, the track which is grounded (having better traction than the water-borne track) will run along the bar. This situation occurs more often when the tractor is retracting than when landing, since the tractor is then less apt to have sufficient momentum to carry it up onto the bar. Both the LVT(A)(5) and the LVT(3)C are susceptible to this condition, but it is more difficult to correct with the LVT(3)C because the differential steering mechanism will not permit the grounded track to be stopped entirely.

If littoral currents exist, they will further complicate the situation because they will be stronger in the troughs between the bars. All these conditions may cause a serious situation if a large breaker should hit the vehicle abeam. Figure VII C-31 shows an LVT(A)(5) in this awkward position. The driver had set a course slightly to starboard to combat the littoral current coming from the right. Suddenly, the tractor was broadside to the incoming breaker and making no headway in the opposing littoral current when the port track grounded. In this case, however, no damage resulted.

An important effect of offshore beach slope is that for given deep-water wave conditions (i.e., for the waves approaching the coast from either a distant or a local storm). The breakers will be higher on a steep beach than on a flat beach; hence, landing vehicles will encounter lower breakers on a flat beach than on an adjacent steep beach, other factors remaining the same.

The effect of the slope of the beach face (Reference VII C-9) is usually negligible for an amphibian tractor because of its excellent trafficability characteristics. However, scarps on the beach face, even if small, present a serious problem. (Figure VII C-32).

(7) Effect of Currents:

Currents, both rip and longshore, have an effect on LVTs. Surf rips can usually be seen at sea and are distinguishable by the large patches of muddy water where the current diffuses seaward of the breaker zone. To make a landing against a surf rip, an LVT has to move at an even lower speed than its normal speed, which is already slow compared to the velocity of waves in the surf zone. On the other hand, surf rips are advantageous to a tractor retracting through the surf. In addition, the breakers are usually lower in a rip.

Littoral currents may cause LVTs to drift off range considerably, but otherwise seem to cause no safety hazards.

As discussed previously, smaller currents may exist at the shoreline if cusps have formed on the beach. Unless information to the contrary has been obtained, the LVTs should land if possible on the point of the cusp and retract off the trough. Although the sand on the points of cusps is less consolidated and generally poorer for trafficability, LVTs will normally be able to traverse it readily.

(8) Effect of Cusps:

The effects of cusps are twofold. One is the effect of rip currents associated with them, and the second is the trafficability problem associated with them.

(9) Variability of Surf:

Surf is extremely variable, both as to position along shore and as to time. Refraction effects due to changes in the alignment of a coast, as well as variable bottom conditions along a beach, may cause marked variation in the character of the surf from point to point along a shoreline. Hence, vehicles coming in may, under certain conditions, be subjected to breakers of various heights. Of more importance for a particular location is the fact that the heights of breakers vary considerably from wave to wave. (Figure VII C-33). Thus a vehicle could get through what would be called a dangerously high surf by approaching or retracting through its groups of low breakers. It is hoped that a method can be developed to discover these low waves a short time before they peak-up and break. This would be most helpful to the Control Officer in directing landings.

Because of this variability, an occasional high wave may break, even during a period of relatively low breakers. For example, on August 4th, 1949, an LVT(3) (without a cargo compartment cover) sank while coming in through the surf off Aliso Canyon. The highest one-third of the breakers was about 4 feet, the maximum breaker height was 6 or 7 feet and the period was about 14½ seconds. A description of this surf casualty has not been obtained, but evidently the tractor encountered one of the very few high breakers.

c. Discussion, Conclusions and Recommendations:

On the basis of studies to date, the following conclusions and recommendations are made:

(i) Experienced Marine Corps LVT officers have not agreed whether, when landing, an LVT should maintain full speed ahead for the entire passage through the surf zone or whether speed should be slackened to jockey for position relative to the wave as it breaks. As has been pointed out the time that a breaker acts upon the craft is doubtless important, and laboratory studies indicate that the distance of a vehicle ahead of the point of breaking is sharply critical. Perhaps if a craft is slowed down at the appropriate time, it will just miss encountering the breaker within the critical area. On the other hand, LVTs underway ride with the bow higher than when dead in the water, thus tending to decrease any forward pitch.

Secondly, if a floating LVT grounds on the tracks with tracks turning, the bow tends to rise, whereas if tracks are stopped, the bow tends to drop. Thus, if the force grounding the bow is from the rear and the LVT is pitched forward, overturning seems less likely with tracks turning. However, during field tests, both an LVT(3)C and an LVT(A)(5) grounded and scraped paint off the bow in an isolated spot well up on the bow, suggesting that the tracks may not have grounded first.

An LVT underway will spend less time in the surf zone than one which travels in intermittent brusts of speed, and will probably be hit by fewer breakers. Finally, an engine turning over at full speed is much more likely to survive foreign particles in the fuel, slight amounts of water over it, or other troubles than one idled down.

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Thus, at present, overall considerations seem to favor maintaining full speed ahead while travelling through the surf zone. It is suggested that operational personnel give some thought to this important item of LVT procedure and experiment with it.

(ii) At present, assault waves attempt to approach the beach on a line parallel to the shore. From the standpoint of LVT casualties, some staggering of the vehicles would be advantageous. If the vehicle waves could be staggered over three ocean waves (less than thirty [illegible] time the first vehicle lands and the last vehicle of the wave lands) the likelihood of losing many of the vehicles at one time would be lessened. If the vehicle were slightly staggered, then even if all were hit by the same breaker, the number of casualties would be lessened because of the effect of vehicle position relative to the breaker, as discussed previously.

(iii) For LVTs the most dangerous type of breaker is the plunging type, with the surging type apparently the least dangerous, and the spilling type breaker being in between the two. For average drivers, the limiting safe height for a plunging breaker appears to be 8 feet; for a spilling breaker, 12 feet; while for the surging breaker, the limiting height has not been determined. Until further information is obtained it would probably be best to consider 12 feet as the limiting surging breaker height.

(iv) The tractors should be driven through the surf at right angles to the breakers to avoid broaching. When a tractor has broached, it is in the most dangerous position from the standpoint of being turned over by a breaker. However, because of the characteristics of these vehicles, they are fairly easy to keep from broaching, the main hazard being that of pitching over. LCVPs and LCMs are quite different, however. Since these craft must maintain their positions on the beach face while discharging troops and cargo, they are continually subject to wave forces which tend to broach or beach them. The LVT, on the other hand, immediately proceeds up the beach upon grounding so that the greater danger is in pitching while traversing the breaker [illegible]

(v) Even though a tractor is not pitched over by a high breaker, loose cargo may be thrown about and be a hazard to the driver and other personnel. Perhaps [illegible] should be installed behind the driver for protection. Also, personnel in the compartment may be thrown about with resulting injuries. Adequate [illegible] should be installed for use by personnel.

(vi) The amphibian tractor was found to be very useful in pulling stalled vehicles across the beach, lifting the stuck ramp of landing craft, and helping to right

broached craft. Accordingly, several special jobs (with a dozer type blade or other modifications) would probably be of great value to the Beachmaster for these miscellaneous jobs.

(vii) The present modified LVT(3)C as well as the LVT(A)(5) is much more "surf-worthy" than the old type LVT(3) because it is not subjected to the swamping problem.

(viii) For a given wave climate, a steep beach (1:10 to 1:15) appears to be more dangerous to LVTs than the moderately flat beaches. A mild or flat beach (1:30 or flatter) is poor because of the very wide surf zone during periods of high breakers. Little is known of the effect of very steep beaches (1:5 or steeper), although limited prototype and model observations seem to indicate that they are not as dangerous as a beach with a slope of 1:10.

(ix) From a trafficability standpoint, LVTs have no trouble on sand beaches unless scarps are present.

(x) LVTs should be loaded in such a manner that the weight is acting as far to the stern as possible. The characteristics of this amphibian vehicle are such that any additional buoyancy at the bow or additional weight at the stern will improve its surf characteristics.

(xi) In regard to the firing of howitzers from an armored LVT, the vehicle is most stable when in the trough of a wave. It should also be remembered that its bow will pitch upward several degrees when on the seaward slope of the breaker so that the angle of depression of the howitzer will be limited.

3. References Cited:

VII C-1: Headquarters, U. S. Army Forces, Pacific Oceans Area, "DUKW (The), Its Operation and Uses", May 15, 1945 (RESTRICTED)

VII C-2: Isaacs, John D. and Willard Bascom, (1947) "Report on Operation and Characteristics of U. S. Army DUKWs in Oceanographic Investigations" - University of California, Department of Engineering, Fluid Mechanics Laboratory, Berkeley, (ME technical report ME-116-266) September 17, 1947, (RESTRICTED)

VII C-3: ONI 226 (1944) - "Allied Landing Craft and Ships" - Office of Naval Intelligence, April 7, 1944, (CONFIDENTIAL)
ONI 226/1 (not dated) - supplement to above (CONFIDENTIAL)

VII C-4: Iversen, H. W., M. J. Larocco, and R. C. Crooke, (1950a) - "An Initial Investigation of the Pitching Motion of Model LVT(3)" - Series 30, Issue 4, Institute of Engineering Research, University of California, Berkeley, California, June 22, 1950, (unpublished). (RESTRICTED)

VII C-5: Iversen, H. W., M. J. Larocco, and R. C. Crooke, (1950b) - "Laboratory Investigations of Breakers, Section C" - Series 30, Issue 5, Institute of Engineering Research, University of California, Berkeley, California, July 7, 1950, (unpublished). (RESTRICTED)

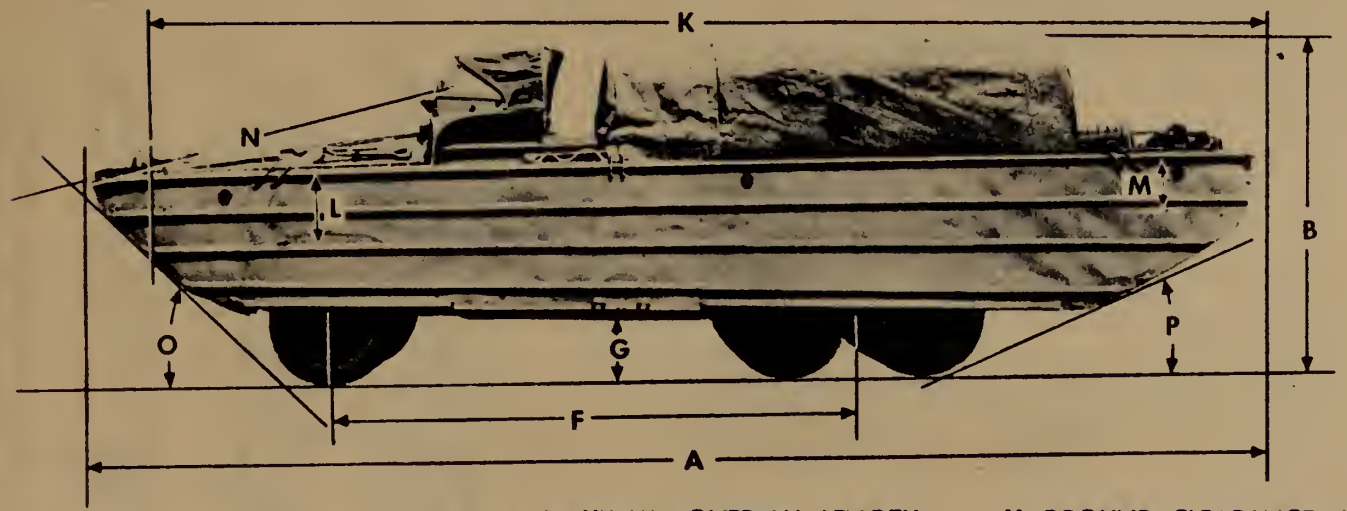
VII C-6: Iversen, H. W., R. C. Crooke and M. J. Larocco, (1951) - "A Study of the Pitching Action of a Model LVT(3)C", Series 29, Issue 43, Institute of Engineering Research, University of California, Berkeley, California, June 22, 1951, (unpublished). (RESTRICTED)

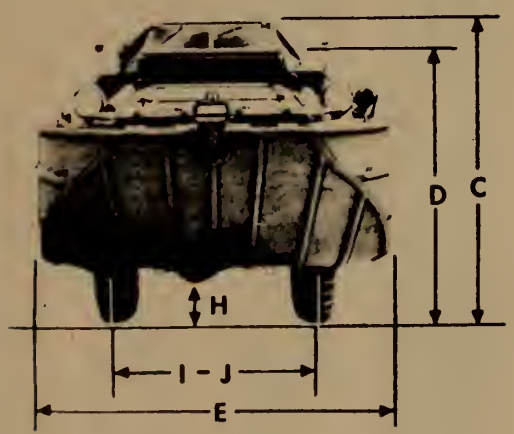
VII C-7: Johnson, J. W. and R. L. Wiegel, (1950) - "Know Your Surf", Series 29, Issue 29, Institute of Engineering Research, University of California, Berkeley, California, September 1, 1950, (unpublished). (RESTRICTED)

VII C-8: Patrick, D. A. (LTJG), and R. L. Wiegel, (1950) - "Qualitative Report on Surf Operations of Landing Vehicles, Tracked", Series 29, Issue 44, Institute of Engineering Research, University of California, Berkeley, California, February 1950, (unpublished). (RESTRICTED)

VII C-9: Wiegel, R. L. (1950) "Trafficability Studies of Selected Beaches on the Pacific Coast of the United States and Oahu, T. H.", Series 29, Issue 30, Institute of Engineering Research, University of California, Berkeley, California, September 15, 1950, (unpublished). (RESTRICTED)

RESTRICTED Security Information




A MAXIMUM OVER-ALL LENGTH—372 INCHES
B MAXIMUM OVER-ALL HEIGHT (TOP UP)—106 INCHES
C MAXIMUM OVER-ALL HEIGHT (TOP DOWN)—98 INCHES
D MAXIMUM OVER-ALL HEIGHT (WINDSHIELD DOWN)—89 INCHES
E MAXIMUM OVER-ALL WIDTH—99 INCHES
F NOMINAL WHEEL BASE—164 INCHES
G GROUND CLEARANCE—HULL—18 INCHES
H GROUND CLEARANCE—AXLE—11½ INCHES
I TIRE TRACK—FRONT—63⅝ INCHES
J TIRE TRACK—REAR—63⅞ INCHES
K LOADED WATER LINE LENGTH—344 INCHES
L LOADED FREEBOARD—TO DECK AT FRONT—24 INCHES
M LOADED FREEBOARD—TO DECK AT REAR—16 INCHES
N LINE OF VISION
O ANGLE OF APPROACH—38 INCHES
P ANGLE OF DEPARTURE—25 DEGREES 15 MINUTES

RA PD 307377

FIGURE VIIC-1 - VEHICLE DIMENSIONAL VIEWS (AFTER DUKW-353-2005)



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a 0820 - CARGO SWEPT OFF AN OVERLOADED DUKW.

b. 0820 - CONTINUAL STREAM OF CARGO - CARRYING DUKWS PLAYING BACK AND FORTH.

c. A LOADED DUKW RIDES THROUGH THE SURF.



d. 0926 - A DISABLED DUKW IS RESCUED BY BEING TIED BETWEEN TWO OTHERS AND BROUGHT TO SHORE.

e. 0929 - DISABLED DUKW SAFELY ON SHORE.

f. 1248 13 NOV. 47 - A DUKW STUCK IN A HOLE IN BEACH.

FIGURE VIIC-2 - DUKWS IN AN AMPHIBIOUS LANDING



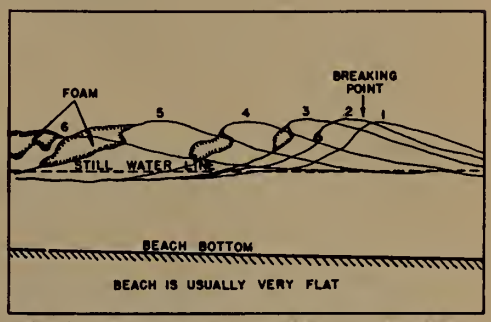
RESTRICTED Security Information

FIGURE VIIC-3 - BREAKER TYPES

Both photographs and diagrams of the three types of breakers are presented below. The sketches consist of a series of profiles of the wave form as it appears before breaking, during the breaking, and after breaking. The numbers opposite the profile lines indicate the relative times of the occurrences.



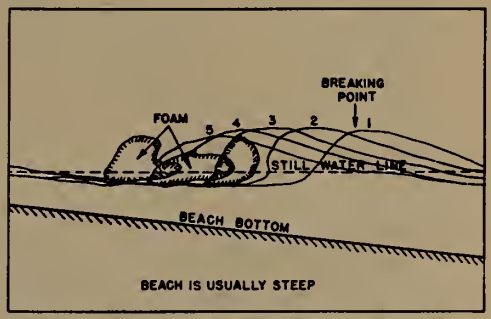
SPILLING BREAKER




SKETCH SHOWING THE GENERAL CHARACTER OF SPILLING BREAKERS



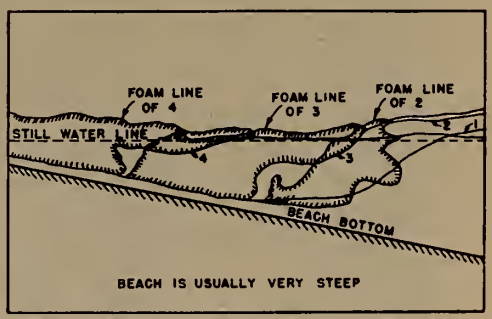
PLUNGING BREAKER




SKETCH SHOWING THE GENERAL CHARACTER OF PLUNGING BREAKERS



SURGING BREAKER




SKETCH SHOWING THE GENERAL CHARACTER OF SURGING BREAKERS



FIGURE VIIC-4 - DUKW PROCEEDING TOWARDS PLUNGING BREAKER

This breaker is about 7 feet high. The driver has selected a course which will meet the breaker head-on, which is the correct procedure. For all practical purposes, a surf of plunging breakers 8 feet high is the maximum that should be attempted in routine work. Under emergency conditions, the DUKW will operate in surf that is plunging 16 feet high. Breaking waves of any type 10 feet high or more are dangerous, and extreme care should be taken. The crew should wear life jackets at all times and take advantage of any condition which will decrease the hazard of the operation.



FIGURE VIIC-5 - DUKW SWAMPED IN SURF

a. This casualty was the result of operating in a dangerous surf without properly securing the protective canvas covering. Several breakers broke over the craft, filling it with water. Then as the craft pitched down over the back side of a breaker, the water ran forward into the engine compartment, shorting the ignition system. The engine stalled and succeeding breakers swamped and sank the craft. The plunging breaker shown above is about 16 feet high. When operating in hazardous surf areas, the DUKW canvas, gasoline cap, engine compartment hatch, and all other appurtenances must be properly secured to make the craft as water tight as possible.



b.. Windshield shattered after encountering several 15-foot breakers in a surf similar to that shown in Figure VIIC-10.





FIGURE VIIC-6 - ILLUSTRATIONS OF GOOD SEAMANSHIP

The driver has selected a course and timed his approach to cross the breaking wave at its lowest height.

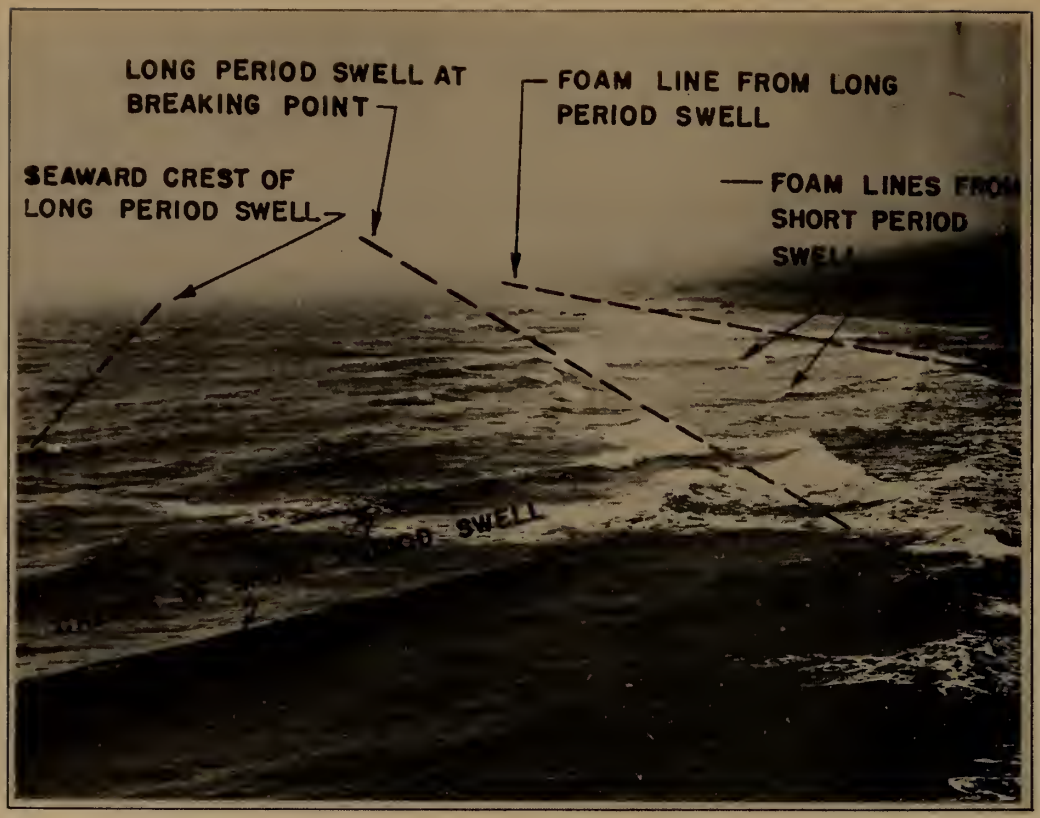
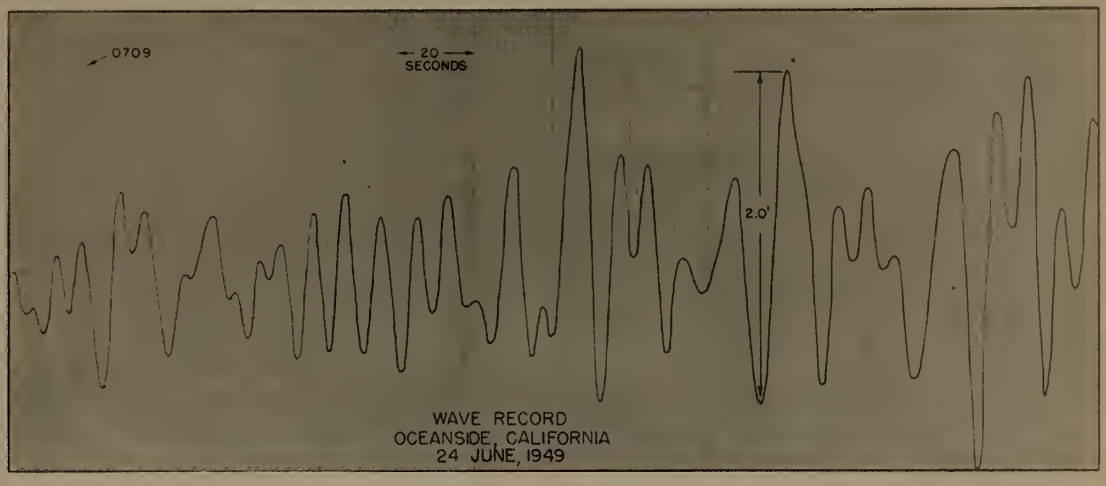



FIGURE VIIC-7 - LONG PERIOD SWELL
AND HIGH SHORT PERIOD WAVES SUPERIMPOSED

This combination produces an unusually difficult surf. The short period swell acts on the long period swell in such a way that it breaks in a surf of oblique sections. A DUKW trying to meet the dominant breakers head-on is subjected to pounding by other portions of the breaker at an angle to its course, which contributes heavily to broaching. In addition, high currents, up to 3 or 4 knots, may be set up which will impose greater difficulties and tend to throw the craft off its course. Furthermore, the point of intersection between two waves results in a high, violent and turbulent breaking crest, flanked on either side by breakers approaching at an angle. A DUKW driver, finding himself between this pair of pincers, is unable to meet either breaker in an advantageous manner. If this pincer position can be avoided, the operator can traverse the surf by changing his direction rapidly and taking each breaker head-on.

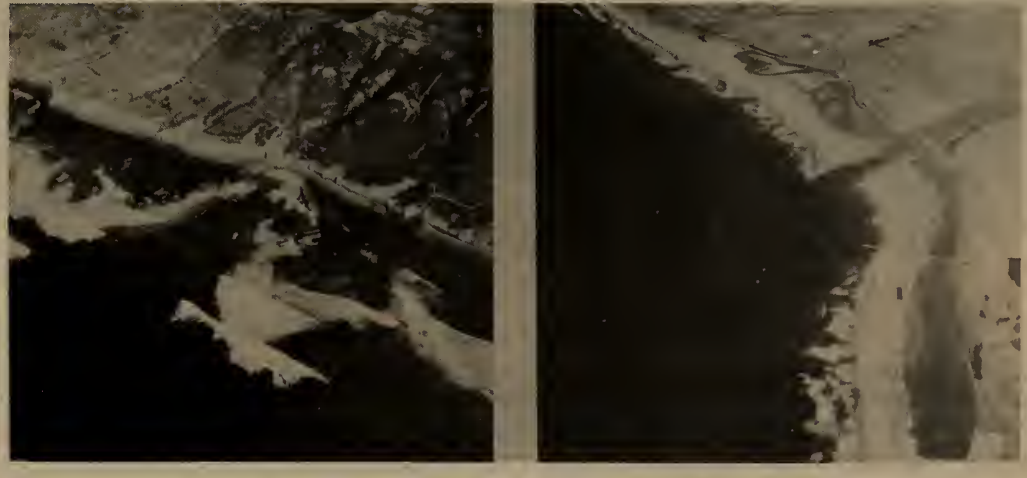
FIGURE VIIC-8 - VARIABILITY OF SURF

Surf is extremely variable both with position along the shore and with time. Refraction effects due to changes in the alignment of a coast, as well as variable bottom conditions along a beach, may cause marked variation in the character of the surf from point to point along a shoreline. Of equal importance is the fact that the height of waves varies considerably from wave to wave. As illustrated by the following chart from a wave recorder, wave trains are usually characterized by groups of high waves separated by periods of relative calm. Waves vary appreciably both as to height and period. If it is desired to traverse the surf in either direction, observe the sequences of the highest waves, then start immediately after the occurrence of a set of high waves because a series of low waves usually occurs in the succeeding minutes.




Typical Wave Record

This record demonstrates the fallacy of the old statement that every seventh wave is a high wave. Passage through the surf should be timed to follow immediately after the occurrence of the higher waves.



Point Loma, California

Moss Landing, California

VARIATION OF SURF ALONG A BEACH

An appreciation of the principles of wave refraction permits the selection of a locality where the surf is relatively low.



FIGURE VIIC-9 - DUKW ENTERING SURF FROM STEEP BEACH

Notice the single foam line. Steep beaches of this nature normally have only one line of breakers. By selecting a period of lull between breakers, and traversing the narrow zone at full speed, the DUKW is able to traverse the surf area without encountering any high breakers and endangering the vehicle.



FIGURE VIIC-10 - SURF AT TABLE BLUFF, CALIFORNIA

Swell spilling on outer bar and plunging on inner bar. The outer bar is so deep that the waves barely reach an unstable condition. They are thus subject to disturbances and will spill. Passing over the outer bar they reform, and, as they encounter the steep inner bar, peak up rapidly and plunge.

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DATE




FIGURE VIIC-11 - DUKW GOING OUT THROUGH AN INTENSE AND COMPLEX SURF

A wide beach with flat slope such as this frequently has several bars which produce a spilling type breaker on the outer bar and a plunging type breaker on the inner bars. This may result in an impassable surf on one of the inner bars, even though the surf is quite moderate on the outer bar. By observing a complex surf zone as shown above, the driver may be able to detect areas of low breakers over the various bars, and select a safe course through the surf area. The DUKW shown above is heading toward a point in the outer bar where the breakers are lower than elsewhere. The driver has selected this course from observations on the beach. The breaker in the upper right is spilling about 18 feet high. This is a dangerous surf.



FIGURE VIIC-12 - DUKW SUCCESSIVELY NEGOTIATES A 15-FOOT PLUNGING BREAKER

The DUKW (circled) was completely covered by a 15-foot plunging breaker, the first of four in a row, which were encountered by the driver. A total of 14 breakers were met during this trip through the 1800-foot wide surf zone (Clatsop Spit, Oregon). A surf like this should be attempted only by a highly skilled and experienced driver for emergency reasons.



a. Landing barge station at Monterey Bay during period of maximum erosion of beach.

1-29-47



b. Same station two weeks later showing extremely rapid changes on this beach.

2-12-47

FIGURE VIIC-13 - EXAMPLE OF BEACH CHANGE



FIGURE VIIC-14 - DUKW STUCK IN SLOUGH

A DUKW operating in quiet water (harbors, bays, lakes, rivers) is not subjected to violent wave forces as are found in a surf zone. The greatest difficulty in areas of quiet water is in finding a suitable site for crossing the shoreline adjacent to the waters edge. Shoreline areas along quiet waters are generally composed of silty or granular and incbhesive materials which do not furnish bearing capacity or traction for DUKW operation. In addition, there are often mud bars offshore. A DUKW moving at five knots will plow through a mud or silt deposit for quite a distance. It will settle down immediately on the hull and offer tremendous resistance to further movement. The DUKW driver should therefore carefully reconnoiter the shoreline area and select the best site before attempting to enter or leave the water.



a. LVT(4)s landing through 3-4-foot spilling breakers.



b. LVT(3)s moving to beach flanks in preparation for retraction.



c. LVT(3)s retracting through 3-4-foot breakers.

FIGURE VIIC-15 - LVT'S IN 3 TO 4-FOOT BREAKERS



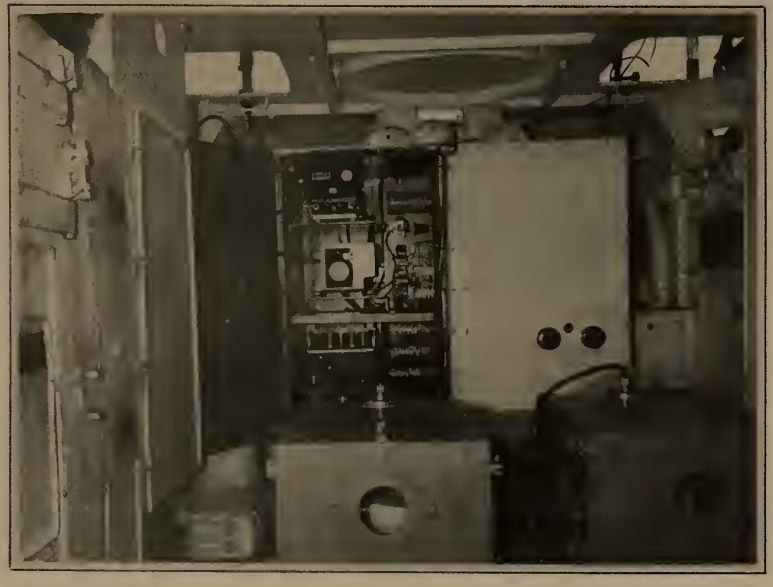


a. LVT(3)C





b. LVT(A)5





c. Radio controlled LVT(3)C

FIGURE VIIC-16 - TEST VEHICLES

SOUTH TOWER, F-56, 8¼"  NORTH TOWER, F-56. 8¼".



FIG. VIIC-17. 38° PITCH ON TRIP IN, RUN #6, 24 FEB. 1950.

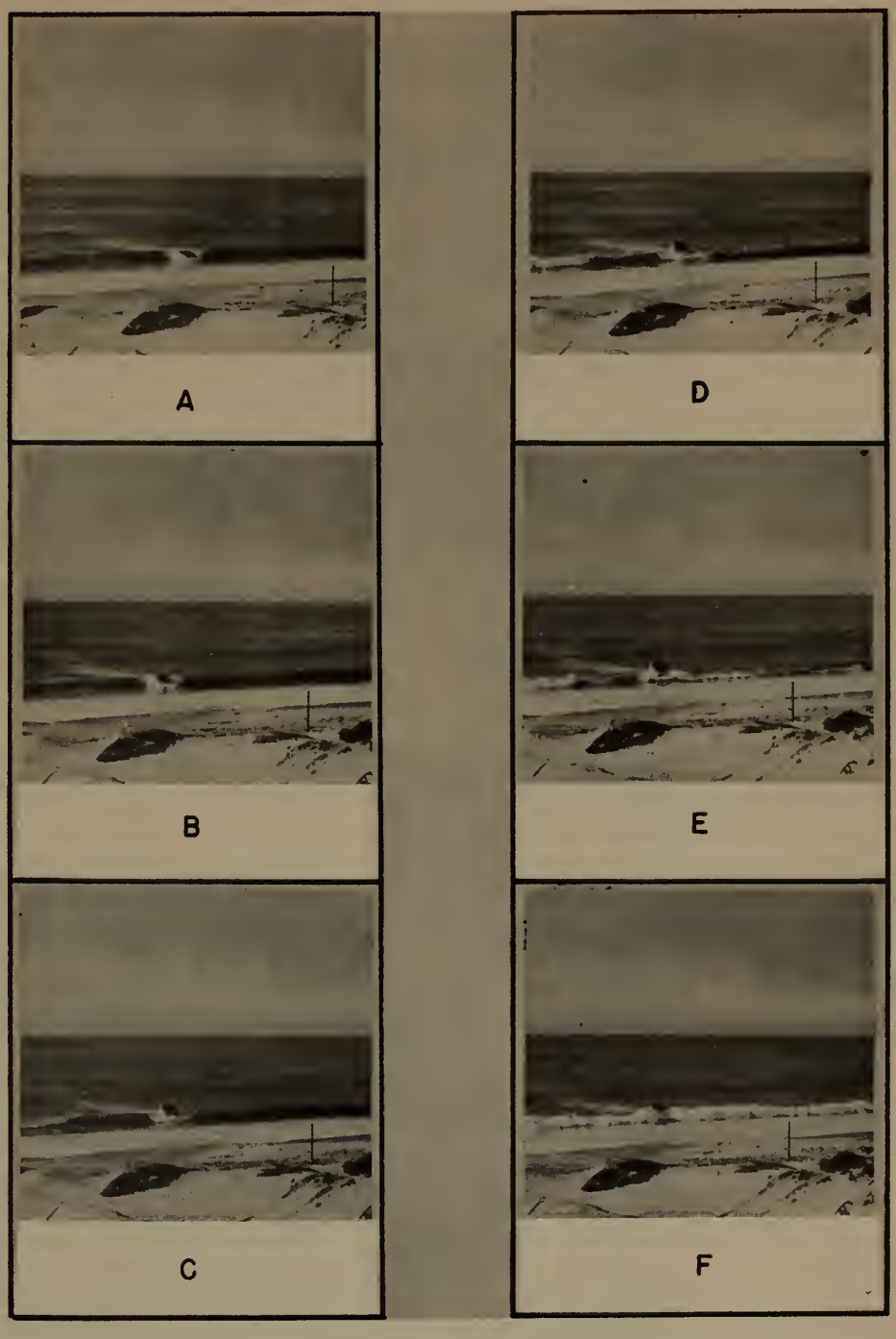



FIG. VIIC-18. PRINTS FROM 35mm. MOTION PICTURE SEQUENCE OF 38° PITCH.




a. LVT(3)C after passing through 8-foot plunging breaker, Tank Farm Beach, 2/24/50.



b. LVT(A)(5) after passing through 12-foot spilling breaker, Clatsop Spit, 5/22/50.

FIGURE VIIC-19 - LVT'S WITH SCRAPED BOWS AFTER PITCHING CRITICALLY WHEN LANDING




FIGURE VIIC-20 - ARMORED LVT IN A 6-FOOT SURGING BREAKER



FIGURE VIIC-21 - LVT(A)(5) retracting. Run 147, Clatsop Spit, Oregon, 5/10/50



FIGURE VIIC-22 - LVT(A)(5) approaching. Run 148, Clatsop Spit, Oregon, 5/10/50



a. 1155:55 - Tractor hit by a 10-foot plunging breaker during retraction. Run 151



b. 1155:56



c. Tractor surf-boarding on a 9 to 10-foot breaker in approach.

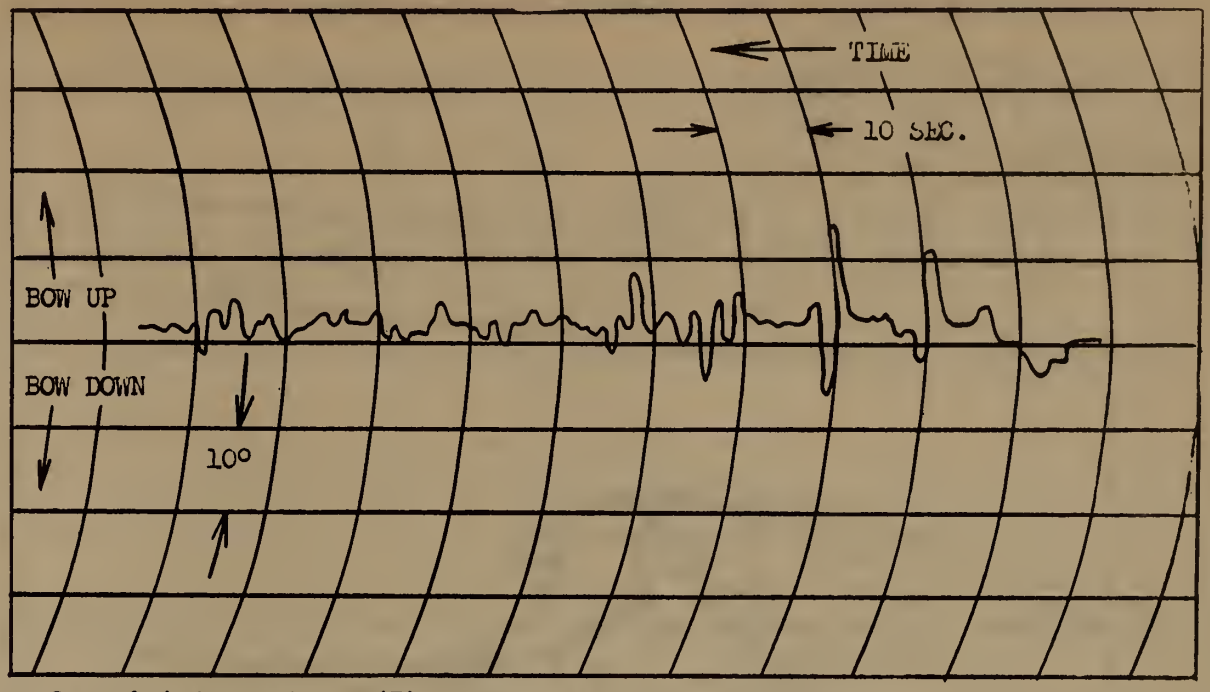


d. Tractor hits breaker on retraction.

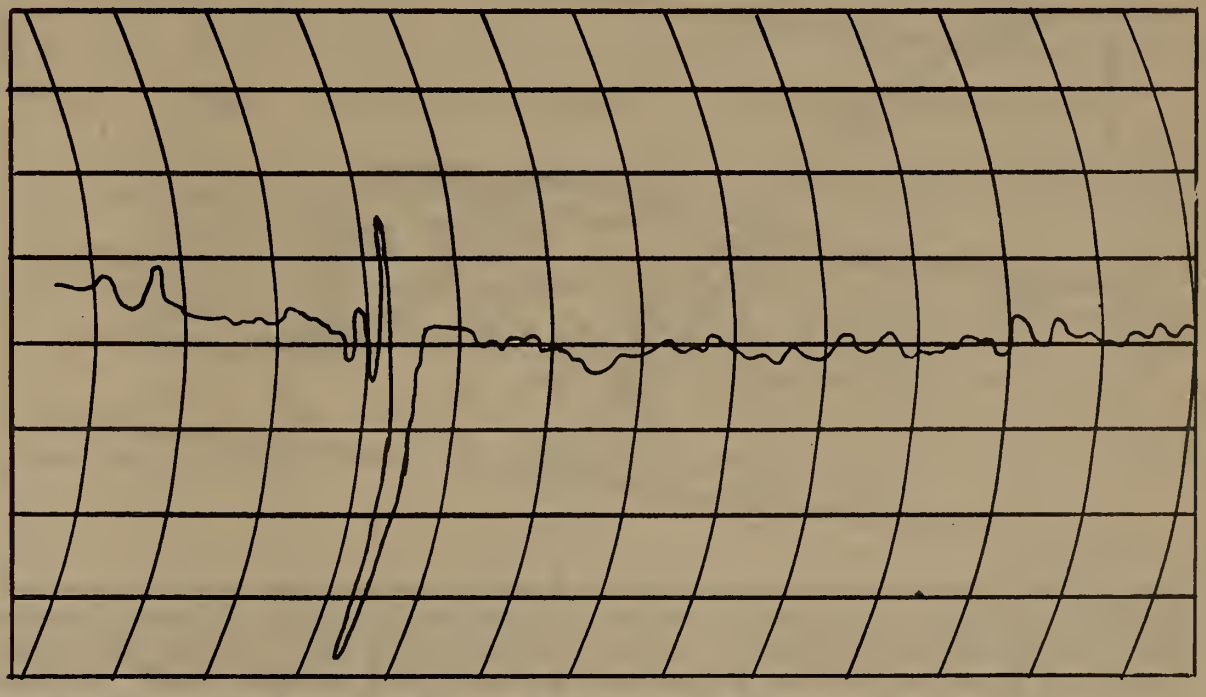
FIGURE VIIC-23 - Amphibious Tractors in Surf, Clatsop Spit, Oregon, May 1950



RESTRICTED Security Information




a. Copy of pitch record, LVT(3)C, Run 6, Retracting, 24 Feb 50



b. Copy of pitch record, LVT(3)C, Run 6, Landing, 24 Feb 50

FIGURE VIIC-24 - COMPARABLE PITCH RECORDS RETRACTING & LANDING

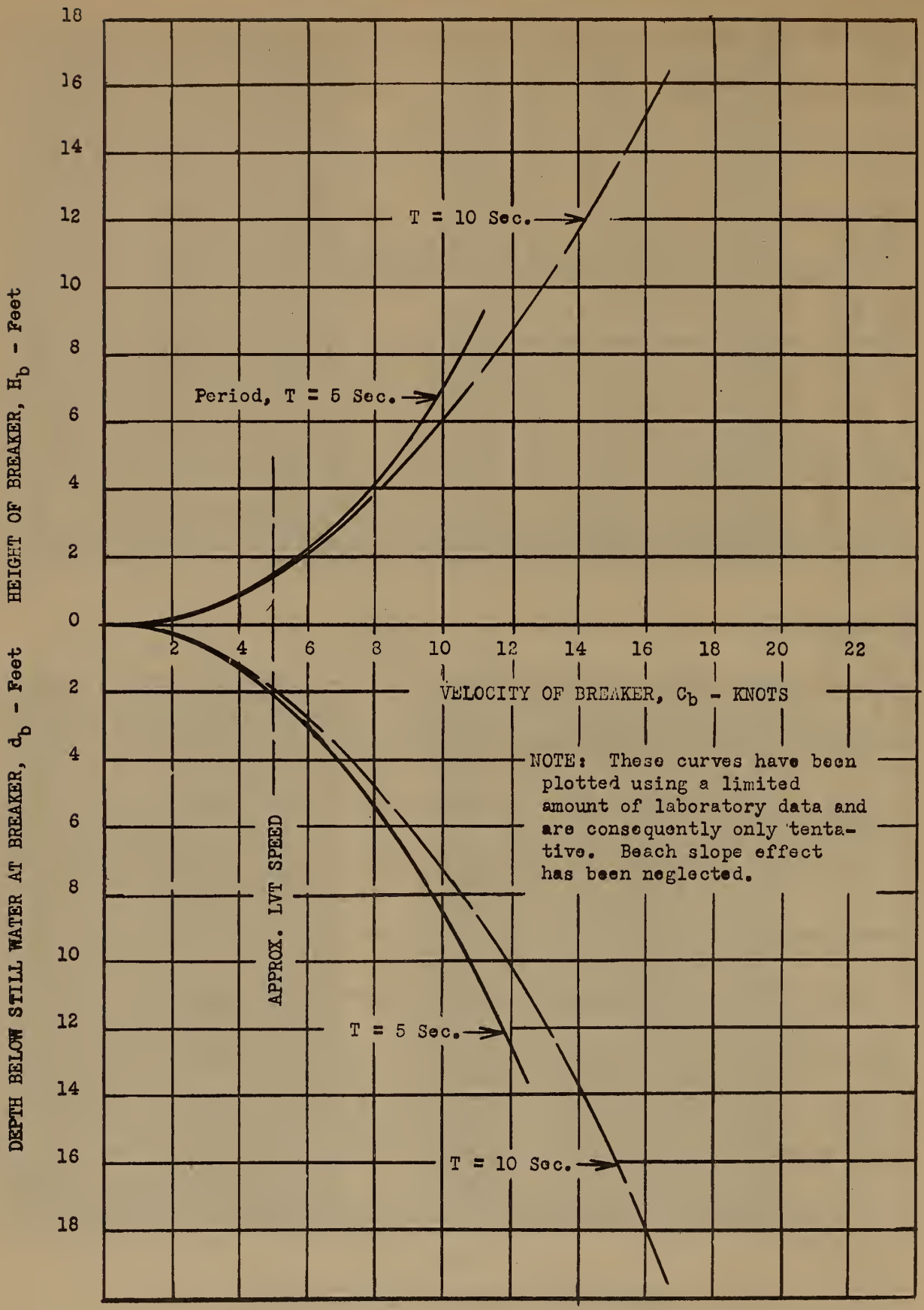
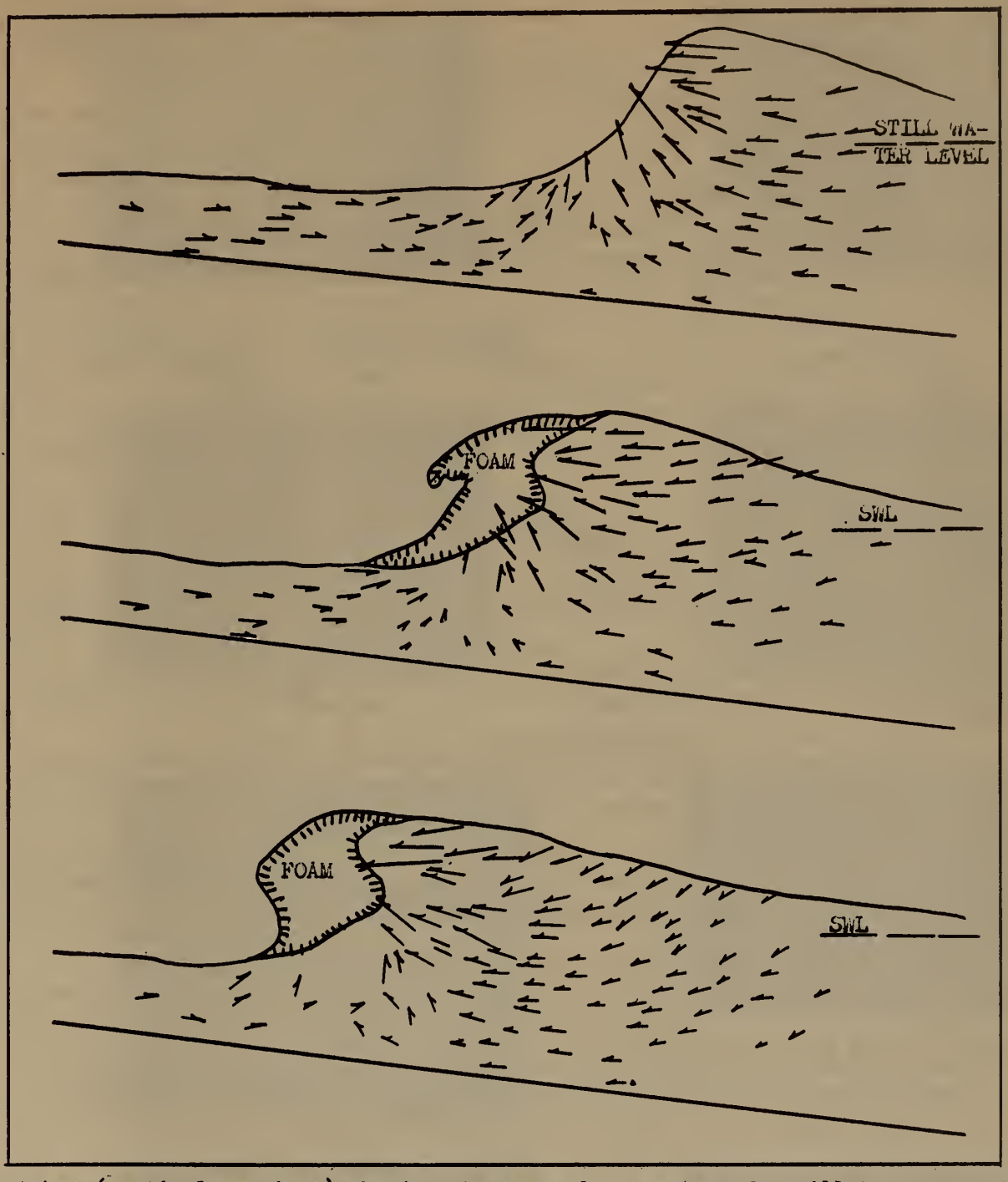
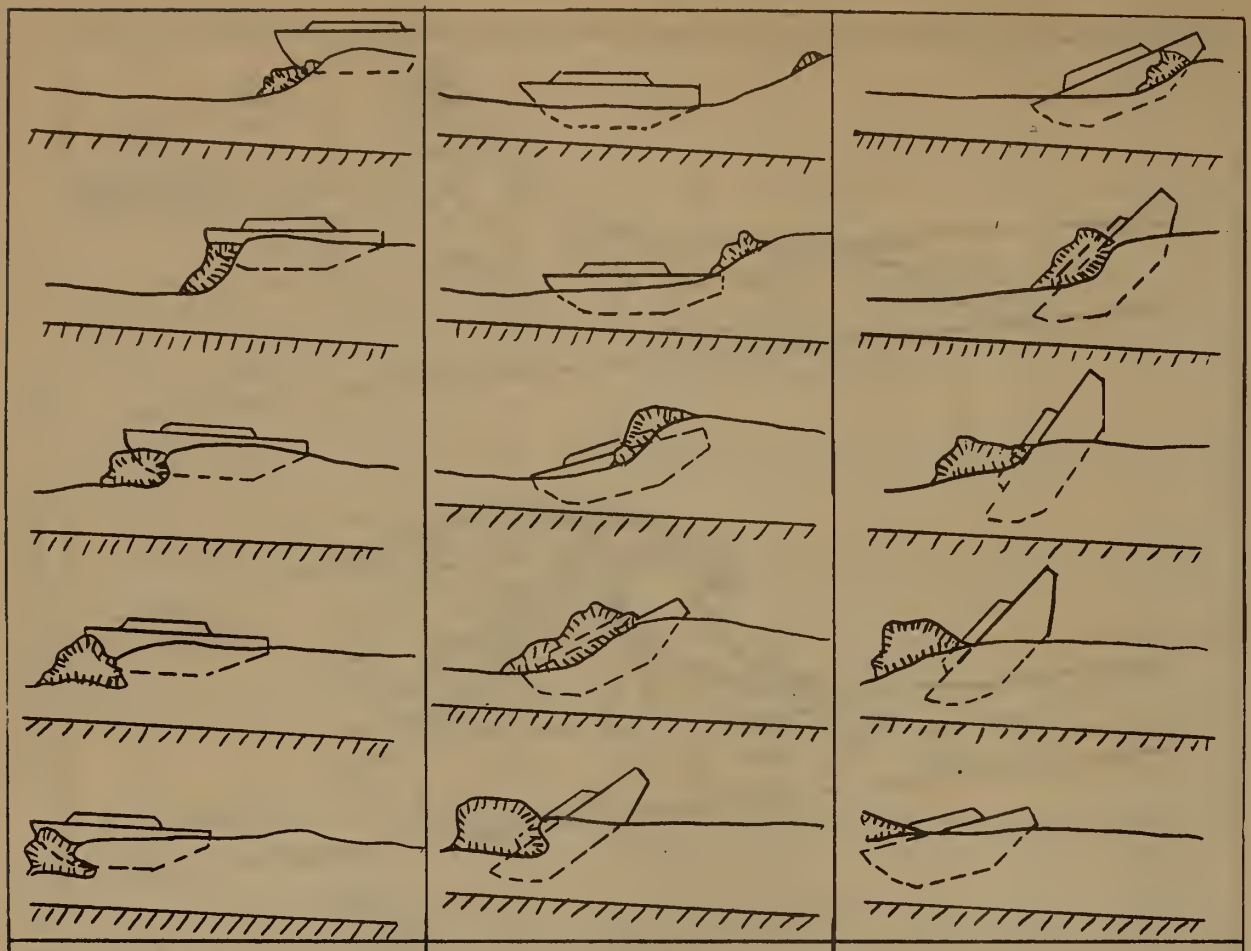



FIGURE VIIC-25 - BREAKER VELOCITY, DEPTH, HEIGHT RELATIONSHIPS




Sketches (vertical sections) showing the general character of oscillatory currents immediately before a wave breaks, at the point of breaking, and immediately after breaking. These sketches were prepared from motion pictures of waves in a glass wall channel. Length of the arrows corresponds to the particle velocity.

FIGURE VIIC-26



| | | |
|---|---|---|
| Surfboards (negligible pitch)<br>CG of craft just behind wave crest at time of breaking. | Maximum pitch, 30°<br><br>CG of craft one craft length in front of wave crest at time of breaking. | Maximum pitch, 52°<br><br>CG of craft just in front of wave crest at time of breaking. |

Prototype conditions: Breaker height, 8.0 feet, wave period, 14½ seconds; beach slope, 1/15.

The position of the center of gravity (CG) of the craft relative to the wave crest at the time it breaks is very important. The sketches shown above (drawn from motion pictures taken of a LVT(A)(5) model in a wave channel) illustrate the great variation in pitching for three conditions. It has been found that craft is in the most dangerous position when the center of gravity of the craft is just in front of the wave at the time of breaking.

FIGURE VIIC-27 - OSCILLATORY CURRENTS - EFFECT ON CRAFT

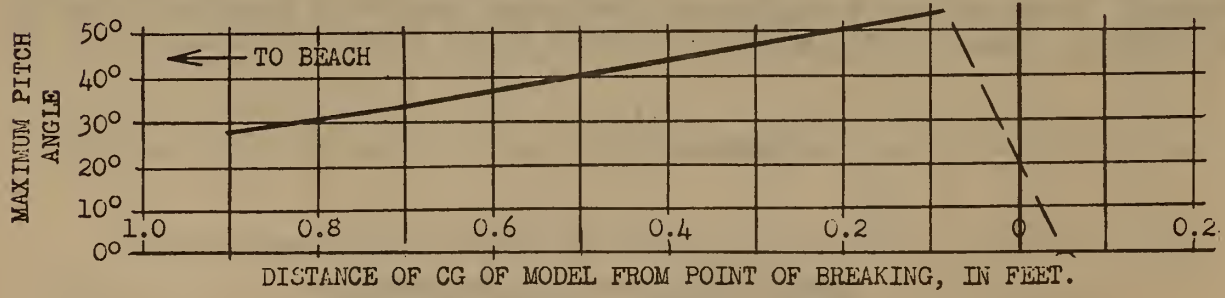
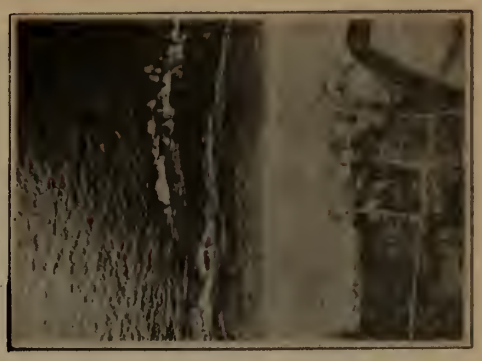



FIGURE VIIC-28



Heavy surf on a flat beach on the Washington coast. Passing through this surf zone requires considerable time; hence exhausting to the operator.

Surf on a flat beach



Light surf at the same locality. Compare widths of the surf zone.



Heavy surf on a steep beach, Ft. Ord.

Surf on a steep beach



Light surf at the same locality.

**FIGURE VIIC-29 - Wide range of beach slopes found on the Pacific Coast**






Flat beach (above), Clatsop Spit, Oregon

Steep beach (below), Monterey Bay





**FIGURE VIIC-30 - Radio-synchronized photographs showing surf conditions on both flat and steep beaches when viewed from the air (left) and from the shore (right). The photos show the same waves. Breakers seldom appear dangerous when viewed from the sea.**



FIGURE VIIC-31 - LVT(A)(5) loses control in trough between bars. Run 150, Clatsop Spit, Oregon, 5/10/50.



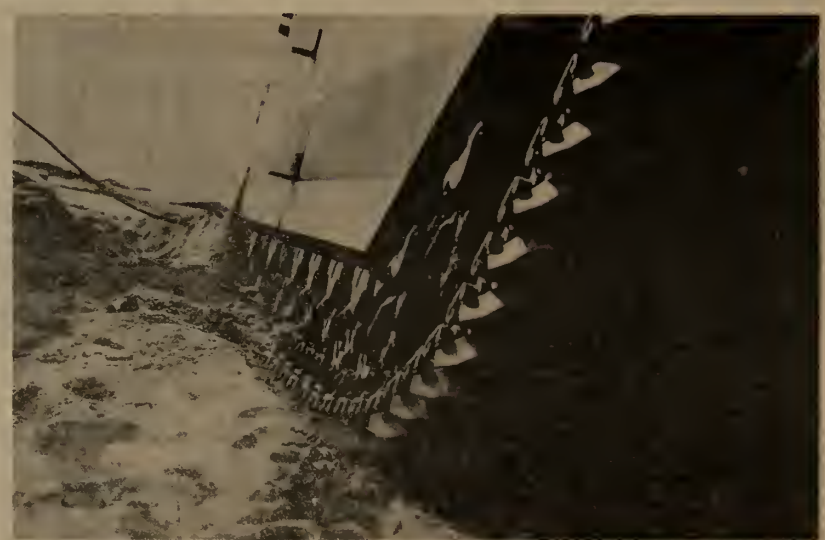
A. LOCALLY GENERATED STORM WAVES AT HIGH ANGLE TO BEACH CUT A 2' SCARP.



B. TRACTOR TAKES THREE TRIES TO NEGOTIATE A SCARP.



C. TRACTOR THROWS TRACK IN ATTEMPT TO GO UP SCARP AT AN ANGLE.



D. THROWN TRACK ON TRACTOR.

FIGURE VIIC-32 - CAUSE AND EFFECT OF SMALL SCARP FORMED DURING STORM OF 10 DECEMBER 1950, SANTA MARGARITA RIVER BEACH, OCEANSIDE, CALIFORNIA

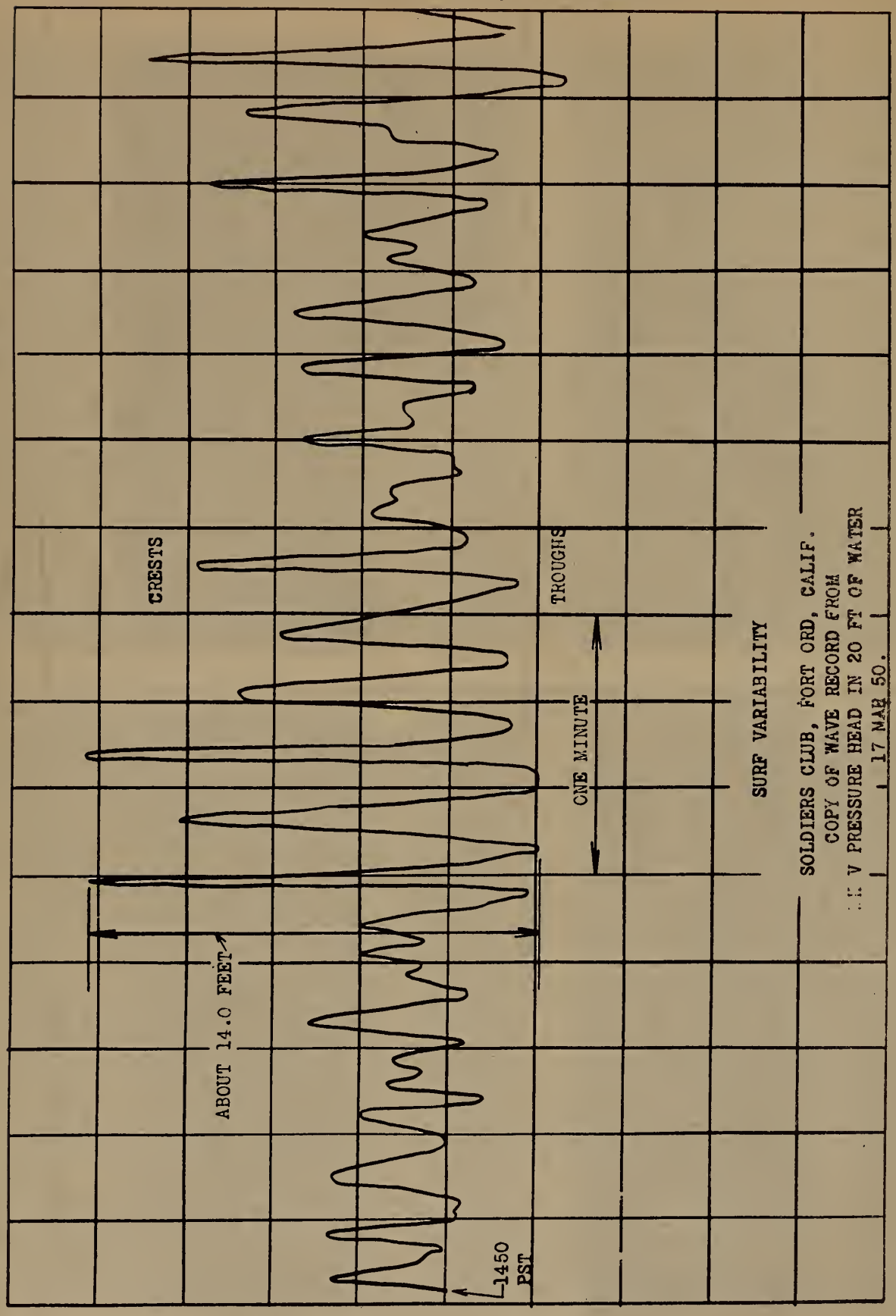



FIGURE VIIC-33

MANUAL OF AMPHIBIOUS OCEANOGRAPHY

SECTION VII. SURF CHARACTERISTICS AND SURFMANSHIP

# D. NAVY LIGHTER PONTOON GEAR

by

Lt. D. A. Patrick, CEC, USN

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"CLASSIFICATION CANCELLED OR CHANGED TO Confidential BY AUTHORITY OF ONR ser 26034 8 Dec 53 (DATE). (SIGNATURE) Mat Brown (RANK) Ow.

## D. NAVY LIGHTER PONTOON GEAR

### 1. Introduction:

Navy Lighter Pontoon Gear has proved its effectiveness for supply phases of amphibious landings. It was used extensively during World War II and also in the Korean conflict. This gear is both versatile and adaptable: while specific units are designed and fabricated for the landings themselves, they often can be salvaged later for use as storage tanks, bridge spans, piers, drydocks, etc. One disadvantage in using them in landing operations is that they may broach or break loose and become a very serious menace to beach traffic and navigation. Despite this danger, they are a valuable and proved amphibious component. (Use of pontoons in invasion harbors is discussed in Part IX of this Manual.)

For optimum use of pontoon structures in any amphibious operation, there should be a specially qualified officer on the planning staff. This is now standard naval procedure. The relationship of any craft or structure to its surroundings is extremely complex, especially in the surf zone. Only an experienced pontoon officer, therefore, can make the final judgments and decisions - and any predictions of probable behavior of this equipment can be proved or disproved only in the actual situation.

The purpose of this report is to summarize for ready reference the available data about amphibious use of pontoon structures. After a brief review of the principal amphibious pontoon structures, there is a discussion of the relationships between these structures and various oceanographic factors. The qualitative effect of many oceanographic factors can be foreseen, and prior study of them will assist in the planning and execution of any amphibious operation.

The limiting magnitudes of oceanographic factors discussed here are considered for each factor separately (i.e., shelter required during storms; effect of beach slope; effect of currents; and effect of wave heights and breaker angles). Of course, these limiting magnitudes may vary for different combinations of these factors.

### 2. Principal Amphibious Pontoon Structures:

Because pontoon structures are made up of small units, a wide variety in the range of sizes is available. The basic unit is the T6B pontoon - 5 feet long x 7 feet wide x 5 feet deep - so that lengths of structure may vary from 5 feet up to 215 feet; and widths from 7 feet up to 56 feet. The nominal size of a pontoon structure is the number of pontoons wide by the number of pontoons long in the deck of the structure. For example, a 3 x 12 barge is 3 pontoons wide and 12 pontoons long. More complete information on the assembly and characteristics of pontoon gear has been published in the Pontoon Gear Manual (1944, VII D-4)*.

Table VII D-1 presents data on the more common pontoon structures. (This shows only a small percentage both of the total number of types; and also of the great variety of uses to which pontoon structures can be put.) Specifically, for amphibious operations in the early phases, the major structures are:

(i) Causeways for ship-shore traffic,
(ii) Barges for ship-shore lightering,
(iii) Warping Tugs for general utility and general salvage operations.

---

* See References Cited in the Appendix.

In the evolution of pontoon structures, these three types have come to be fabricated in specific sizes, as discussed below.

a. 2 x 30 Causeway:

A causeway "set" consists of two 2 x 30 causeway sections which are flat barge structures with a ramp on the end of one of the barges to permit traffic over the causeway to the beach (Figure VII D-1). Total length of causeway (composed of a number of sets) is governed by: the quality of the nearby anchor-holding ground; protection of the structure from wind, waves, and the elements; and the number of causeway sets available.

In normal use, the causeway units are joined by a lap splice of about 25 feet of their length so that the maximum effective length of two causeway sections (each 175 feet long) is 325 feet. An end-to-end connection is being developed at the Naval Civil Engineering Research and Evaluation Laboratory, Port Hueneme, California, but to date this connection has not been approved for general field use. If a suitable end connection could be developed for the present sections, the effective length would be increased to 350 feet per set. However, to provide flexibility of length where the causeway structure joins the ship, one or more lap connections might still be desirable near the offshore end.

Causeway sections are not self-propelled and each section is usually supplied with one 5,000-pound Navy stockless anchor. Thus, placing and maneuvering of the sections is all but impossible without the aid of auxiliary power. Because the causeway sections are not self-propelled, two methods for the initial beaching have come into practice. One method, first used in action at Sicily, was called "momentum" beaching:

First Method: a causeway set is secured by towing hawsers to the side of an LST and the ship makes its run to the beach. As the LST grounds, the causeway set is cut loose so that it continues to the beach under its own momentum. A tag line is retained aboard the LST so that after the set grounds, the offshore section can be winched back for joining (marriage) to the ship.

Second Method: the sections are placed in position, usually by a pontoon warping tug. When the warping tug is used, power can be maintained on the causeway set even after it grounds, so that often the onshore end can be worked farther up onto the beach. Figure VII D-2 shows a warping tug maneuvering a second causeway set to form a longer causeway.

After the causeway set is placed, the first vehicle off the parent LST is a bulldozer, which is part of the causeway equipment. The bulldozer proceeds ashore to help secure the shore end of the causeway to better sustain action of traffic and waves. Figure VII D-3 shows vehicles disembarking from a LST over a causeway, and Figure VII D-4 shows vehicles coming ashore from the causeway.

One of the greatest disadvantages in using pontoon causeways is that they may broach unless they are well maintained and unless sea and weather conditions are moderately calm. Normally, the loss of the pontoon structures in itself is less important than the fact that they litter the beach. Because of their fairly long lengths, they can block traffic over a large frontage of beach. Figures VII D-5 and 6 show some causeways in use and some broached during the Luzon Operation in 1945.

A single causeway set probably could be maintained in position in 6-foot breakers and be used for limited foot traffic under these conditions. The upper limit for vehicular traffic, however, is about 3 feet, and successful marriage of the causeway to an LST would be doubtful in waves over 3 feet high. Longer causeways, composed of more than two causeway sections, would be more hazardous in breakers much lower than 6 feet, especially if accompanied by currents.

b. 6 x 30 Rhino Barge:

These units were used at the Normandy operation so that ships would not need to beach in the early phases of the landings. Initially, rhino barges were married to LSTs with their sterns to the bows of the LSTs at positions some distance off the beach (Figure VII D-7). They served as conventional automobile ferries; i.e., the vehicles could be driven off directly without turning around when the barge beached. Their capacity was about 40 vehicles, or 500 tons of general cargo, or about one-half of an LST load (Figure VII D-8).

These barges have two Diesel propulsion units and are often aided by one or more 2 x 7 pontoon rhino tugs. The rhino barges were expected to marry effectively to the LSTs in waves not over 3 feet high. However, off Omaha Beach, successful marriages were made in waves 6 feet high, although these circumstances were hazardous and much of the special gear required for the marriage was damaged (Building the Navy's Bases in World War II, 1947, page 111, VII D-1).

During the storm off the Normandy beaches of June 19-22, 1944, the rhino barges were somewhat damaged (primary damage was to their outboard propulsion units). Rhino ferry service was resumed on the first day after the storm at Utah Beach; but at Omana Beach, not until 10 days after the storm. When the barges were no longer required for ferry service from landing ships, they were used as lighters between cargo ships and shore.

The importance of the rhinos is shown by some statistics: 36 rhinos were provided for the Normandy operation, and by D day-plus-10 they had landed 5,286 vehicles at Utah Beach, and 16,000 at Omaha Beach (Building the Navy's Bases in World War II, 1947, page 117. VII D-1).

The Normandy operation illustrates a wide range of operating conditions for the rhinos. On the basis of this operation, some tentative conclusions can be drawn:

(i) Maximum wave height for successful marriages of rhino barges to LSTs is about 6 feet;

(ii) Rhinos seem unable to survive storms unless in protected anchorages.

(iii) When damaged, the main damage seems to be to their propulsion units rather than to the barges themselves.

c. 3 x 12 Ramp Barge:

These barges can be used for: cargo lighters, bowser barges (for refuelling and rearming LVTs and other small craft), floating storage, crane transfer barges, and casualty transfer barges alongside evacuation LSTs. The same ramp section (14 feet wide) is provided as for the 2x30 causeways to

facilitate handling cargo and vehicles at the beach. Figure VII D-9 shows two 3x12 ramp barges lashed together to form a single 6x12 barge.

These barges were particularly effective used as casualty transfer platforms alongside LST Hospital ships, especially in moderately heavy seas. Casualties could be transferred from the small boats and LVTs to the pontoon barge, thence to the deck of the ship.

Special advantages of these units are their resistance to capsizing and their ability to stay afloat in spite of extensive hull damage and failure of the propulsion unit. Thus, they are satisfactory for use as floating storage and standby units for call loads to the beach (Figure VII D-10).

Iseley and Crowl (1951, page 251. VII D-3) report that the need for relieving beach congestion by establishing control and transfer points offshore was first recognized at the Tarawa Operation in 1943. Also, the idea originated there of using floating stocks of various supplies which were available for delivery to the beach on call of the beachmaster. Congestion at the beach can be relieved further by having small craft, particularly LVTs, refuel and rearm from the bowser barges (Figure VII D-11) as at the Tinian Operation in 1944 (Iseley and Crowl, 1951, page 365. VII D-3).

Where cargo must be handled over a reef in changing from small boats to LVTs, crane barges can handle some of the heavier transfers and also provide working platforms for the transfer. The Okinawa invasion of 1945 illustrates the utility of pontoon barges. During the first month of operation, with a total of 92 barges (including several warping tugs), over 25,000 tons of miscellaneous cargo were lightered by cargo barges and over 40,000 tons were transferred by crane barges. (Building the Navy's Bases in World War II, 1947, page 404. VII D-1).

Disadvantages: these barges are quite awkward and helpless in storms, unless there is a sheltered anchorage for them. When there is no sheltered anchorage available (as at Iwo Jima), the barges - and also other pontoon structures break loose and broach easily or float free. They then cause beach congestion, or become floating hazards to navigation.

In the latter case, their otherwise excellent buoyancy characteristics become liabilities. They are hard to sink, even by gunfire, and may stay awash even when most of their outboard pontoons are flooded. Their 200-pound Danforth anchors (one to each barge) are relatively ineffective for this size barge. However, larger anchors would require heavier handling equipment than the present light hand-winches, so that substituting larger anchors would also be somewhat unsatisfactory.

d. 12-13-12 Warping Tug:

These units are very versatile and useful both as tugs and for general salvage use. Equipped with 2 outboard Diesel propulsion units, a 2-drum gasoline powered winch, a large A-frame and one or two 5,000-pound stockless or 3,000-pound Danforth anchors, they are effective as salvage units (Figure VII D-12). With the outboard propulsion units, the screw can be rotated 360° so that full power can be supplied either fore or aft in addition to the line pull of the winch. Figure VII D-13 shows a warping tug placing anchors for causeway sections.

3. Relationships Between Oceanographic Conditions and the Use of Pontoon Gear:

a. Shelter Required During Storms:

Once the pontoon gear has been launched, it cannot be retrieved aboard ship as easily as can small craft and amphibian vehicles. Consequently, unless there is almost no chance of storms occurring during the time of the operation, every effort should be made to select in advance a suitable sheltered area near the operation beach. Such an anchorage is desirable in any event because ships often do not have time to retrieve even their boats when a storm blows up suddenly. Unless substantial moorings are available, pontoon units are likely to break loose. If they are grounded, they clutter up the beach. If they float off, they are hazards to navigation. Hence, suitable refuge anchorage is practically mandatory.

b. Effect of Beach Slope:

The slope of the beach may determine the extent to which pontoon gear will be used. For some of the steeper beaches, pontoon causeways may not be required. However, in a large number of cases, particularly if sand bars or reefs are present, LSTs may not be able to discharge directly onto the beach.

As to the pontoon units themselves, ordinarily the causeways will beach in from 1½ to 2½ feet of water. Since the bilge slope of the causeways is zero in quiet water, the slope of the beach can only affect the length of causeway required. The barges and warping tugs (being equipped with outboard propulsion units which can be elevated while power is maintained to the screw) are not limited by the draft of the screw, but by the draft of the bilge.

The beach slope affects them according to how they are loaded. With the stern one foot deeper in the water than the bow, the slope of the bilge will be about 1/70. Depending on the type of cargo carried, and on whether the amount of the load is limited by maximum weight or simply by the volume capacity of the barge, the barge can be adjusted to the required trim. The steeper beaches are more desirable for the point of touchdown: the flatter beaches may limit the amount and distribution of the load which can be carried.

Retraction of these units is made easier because the full power of the screw is available by reversing the direction of the propeller axis, (but keeping the propeller in forward gear): and also because the slip stream of the screw is more effectively directed up under the barge.

c. Effect of Currents:

Strong littoral currents greatly hamper the use of pontoon units because the units are relatively large and cumbersome. Causeways are particularly difficult in currents, as they tend to orient broadside to the current because of their great length-to-width ratio. The barge units, particularly when loaded, have such slow speeds (about 3 to 5 knots) that currents limit their maneuverability.

d. Effect of Wave Heights and Breakers at an Angle to the Shore:

If these currents are caused by waves breaking at an angle to the shore, the problems of operating the pontoon structures in the surf will be increased. In general, short period waves from local storms will be most hazardous, because they are less affected by refraction before they break, and also because their wave fronts are less likely to be parallel to the shore.

4. Summary:

Thoughtful advance study of the probable oceanographic conditions during a planned operation will guarantee more efficient use of the pontoon gear. This advance study should include:

(i) Consideration of refraction and diffraction diagrams for the area; and also of the forecasted waves and weather for the time of operation. This may disclose sheltered areas not otherwise apparent, or it may show that areas believed to be sheltered actually will not be.

(ii) Consideration of the effect of beach slope. This may allow instructions to be issued beforehand as to distribution and amount of loads (in terms of trim and draft) which can be carried.

(iii) Consideration of currents. This may help in locating the debarkation area, and also in locating holding areas for floating storage and call loads.

(iv) Consideration of wave heights to be expected, and also the estimated angle of the breakers to the beach. This may influence the anchorage pattern and method of placement of the causeways, and also the beaching patterns of the barges.

5. Appendix:

a. References Cited:

VII D-1. Building the Navy's Bases in World War II (1947), Vol. II, Bureau of Yards and Docks, Department of the Navy, Washington, D. C.

VII D-2. History of 302 Naval Construction Battalion (1944 and 1945), Author, publisher, etc. unknown.

VII D-3. Iseley, J. A. and P. A. Crowl, (1951), The U. S. Marines and Amphibious War, Princeton University Press, Princeton, N. J.

VII D-4. Pontoon Gear Manual, (1944) Revised edition, Bureau of Yards and Docks, Department of the Navy, Washington, D. C. (RESTRICTED). (This Manual now being revised.)

TABLE VII D-1

Summary of Characteristics of Some Navy Lighter Pontoon Structures*

| Pontoon Structures | Nominal Size | Deck Dimensions (Feet) | Approximate Displacement (Tons) | Capacity (Tons) | Draft (Feet) Light | Loaded | Propulsion Units | Transport to Objective |
|---|---|---|---|---|---|---|---|---|
| Causeway Section | 2x30 | 14x175 | 80 | -- | 1.5 | -- | 0 | LST Side Carry |
| Rhino Ferry | 6x30 | 43x175 | 200 | 500 | 1.5 | 4.0 | 2 | Tow or assemble at site |
| Rhino Tug | 2x7 | 14x43 | 17 | -- | 2.0 | -- | 1 | Deck Load |
| Barge, Ramp | 3x12 | 21x72 | 50 | 75 | 1.5 | 4.0 | 1 | LST Side Carry |
| Barge | 3x7 | 21x43 | 30 | 50 | 1.5 | 4.0 | 1 | LST Side Carry |
| Crane Barges | 3x7 | 21x43 | -- | 5 | -- | -- | 0 or 1 | Assemble at site |
| | 4x7 | 28x43 | -- | 10 | -- | -- | 0 or 1 | " |
| | 5x12 | 35x72 | -- | 20 | -- | -- | 0 or 1 | " |
| Warping Tug | 12-13-12 | 21x72 | 65 | -- | 2.0 | -- | 2 | LSD |
| Dry Docks | 4x15 | 28x87 | -- | 100 | -- | 20** | 0 | Tow or assemble at site |
| | 6x18 | 43x105 | -- | 250 | -- | 20** | 0 | |
| | 30x12 | 215x56 | -- | 800 | -- | 20** | 0 | |

* This list is far from complete, but illustrates normal ranges in sizes. Variations within these ranges may be made if required.

** Maximum draft over deck when flooded.

Source: Most of these data adapted from Pontoon Gear Manual, 1944, Reference VII D-4.



Figure VIID-1 - A causeway set consisting of two 2x30 pontoon causeway sections.



Figure VIID-2 - Warping tug placing second causeway set in position with beached causeway set for additional causeway length.



Figure VIID-3 - Vehicles debarking from LST over causeway.



Figure VIID-4 - Vehicles coming ashore from causeway.



Figure XIID-5 - Use of pontoon causeways for invasion of Luzon, P.I., January 1945 (from *History of 302 Naval Construction Battalion*, 1944, 1945)

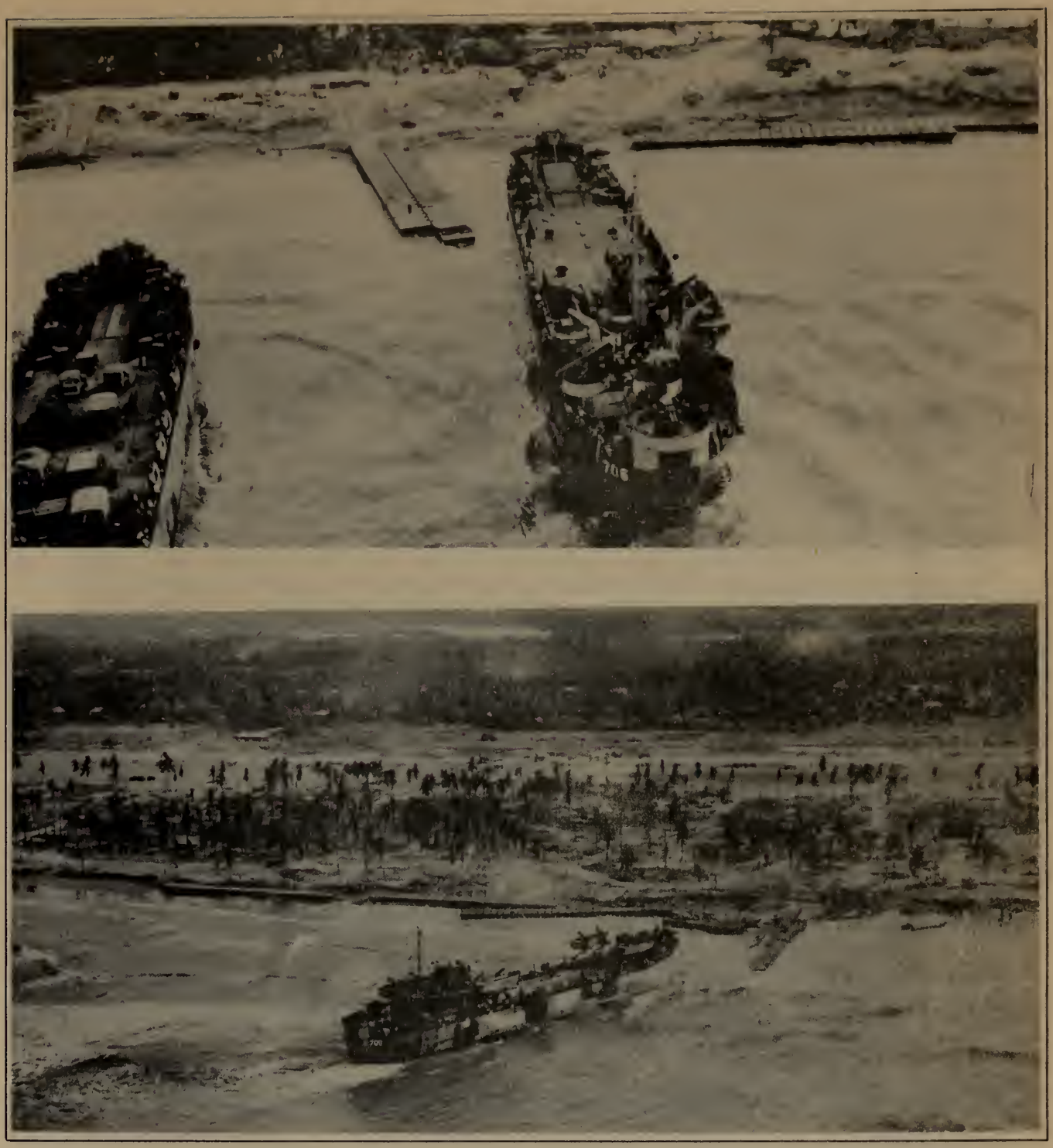


Figure VII D-6 - Some broached pontoon causeways at invasion of Luzon, P.I., January 1945 (from *History of 302 Naval Battalion, 1944 & 1945*)



Figure VIID-7 - Rhino ferry "married" to an LST (from *Building the Navy's Bases in World War II*, 1947)



Figure VIID-8 - Rhino ferries in action at Omaha Beach (June 1944). Loaded rhino coming in to beach, emptied rhino (foreground) being repaired on return trip; DUKW between ferries. (from *Building the Navy's Bases in World War II*, 1947)

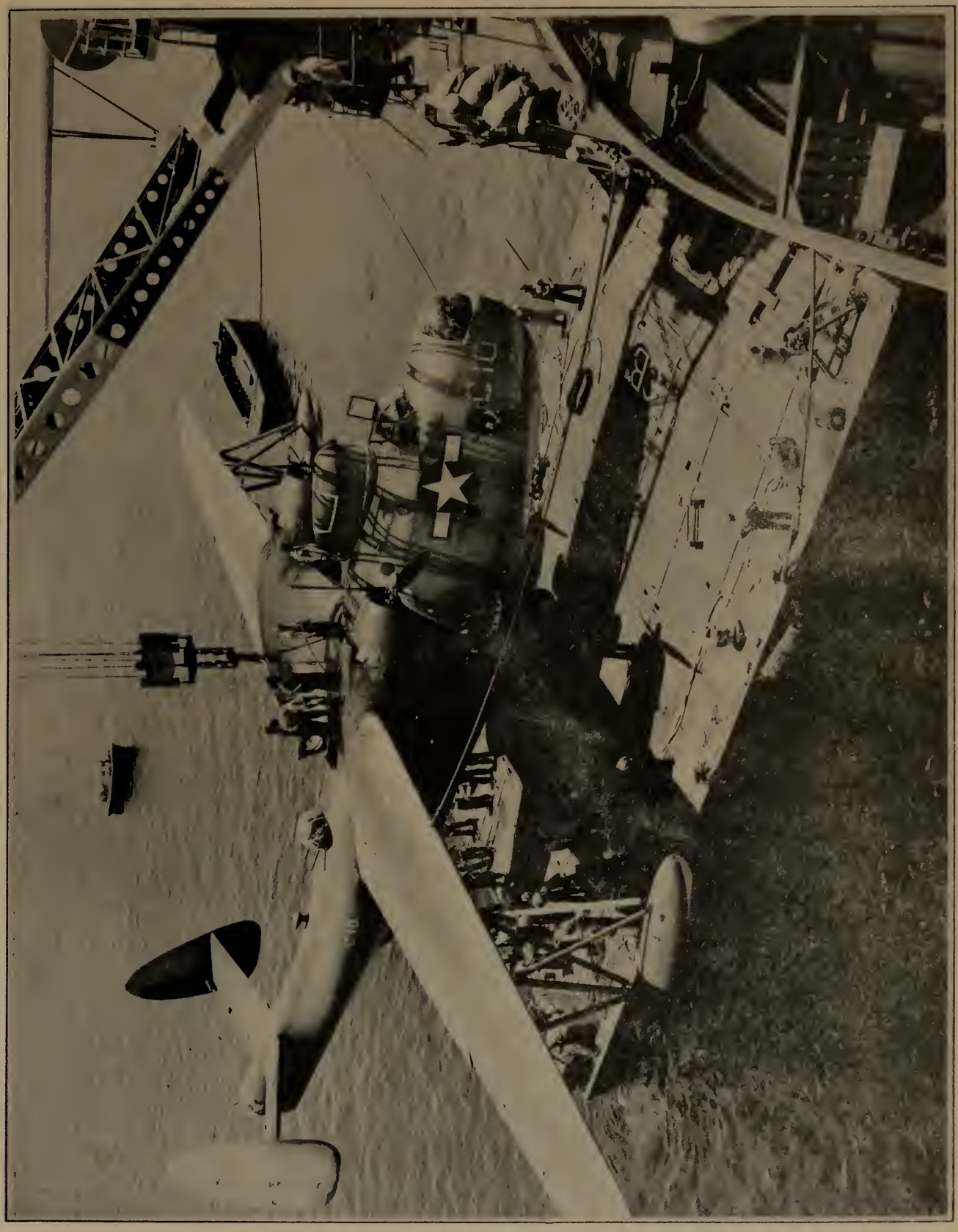


Figure VIID-9 - Two 3x12 ramp barges lashed together to form a single 6x12 barge.



Figure VIID-10 - CB barge loaded with ammunition, Saipan, 1944. (from *History of 302 Naval Construction Battalion, 1944 & 1945*)



Figure VIID-11 - LVTs refuel from CB pontoon barge off Tinian, 1944. (from *History of 302 Naval Construction Battalion, 1944 & 1945*)



Figure VIID-12 - 12-13-12 warping tug anchored in surf.



Figure VIID-13 - Warping tug placing causeway anchors; virtually no surf.

# MANUAL OF AMPHIBIOUS OCEANOGRAPHY

## SECTION VII. SURF CHARACTERISTICS AND SURFMANSHIP

## E. RUBBER BOATS

by

LTJG. D. A. ELLIS, CEC, USN

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## E. RUBBER BOATS

### 1. Introduction:

The use of rubber boats in amphibious operations is generally limited to landing of small raiding parties, beach and surf reconnaissance (Figure VII E-1), underwater demolition team operations (UDT), (Figure VII E-2 and 3), sabotage forays, or other special small scale tactics involving few personnel and small quantities of equipment and supplies.

### 2. Surf Operations:

Breaker height, width of surf zone, and littoral currents are the dominant surf characteristics affecting rubber boat surf operations. Observations of 150 sorties through the surf indicate the maximum safe operational breaker height limits to be 6 to 7 feet, and lower limits are desirable. A comparison* of percent casualties versus breaker height is shown in Figure VII E-4. (These data were for crews undertaking their first training. Skilled crews would not have as many casualties.) It is interesting to note that nearly twice the number of casualties occurred during retractions as during approaches. Discussions with personnel who participated in actual reconnaissances during World War II indicate that this observation is generally true. Excessive retraction casualties result because the breaker tends to pitch the bow of the boat up and over. Even if the boat is not capsized, a man is often thrown overboard by this action.

The width of the surf zone plays a very important part in rubber boat surf operations. A narrow surf zone, say of 200 feet width, will normally possess only one or two lines of breakers. A sortie through surf on a beach of this nature may be accomplished in a few minutes without exhausting crew members. Danger of capsizing is also lessened due to the few number of breakers encountered. A wide surf zone of 1500 to 2000 feet imposes more difficult conditions (Figure VII E-5). Offshore bars and secondary wave trains may cause upwards of 6 or 7 lines of breakers and in moderate seas the entire surf region may become so turbulent and choppy that it becomes difficult for crewmen to distinguish one line of breakers from another. Sorties through surf of this nature are time consuming and exhaustive to personnel. The increased number of breakers present, excessive time required for traversing the surf area, and exhausting effect on personnel, all lead to higher percent casualties.

Longshore currents, when encountered, cause more trouble on wide surf zones than on narrow. The prolonged period required for traversing the wide surf zone provides greater time duration for currents to affect the craft. As a result drifting occurs and personnel must expend additional effort to keep the boat on course. In addition, a wide surf zone, having bars or irregular bottom contours, may have variances in current direction and velocity, which add to the difficulties. Currents in a narrow surf zone are normally of fairly constant magnitude and direction and, if known, allowance for drift may be estimated prior to entering the surf area.

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* VII E-1: Wiegel, Robert L. - "Rubber Boat Surf Casualties" - Series 29, Issue 37, Institute of Engineering Research, University of California, Berkeley, California, 18 October 1950 (unpublished).

Figure VII E-6 is an aerial photograph of surf rips on a steep beach. The current flows seaward as indicated by the arrows. Rubber boat teams, as well as swimmers should take advantage of these currents in going from the beach through the surf and should avoid them when heading into the beach.

3. Equipment:

The use of power driven rubber boats in World War II indicates that gas engine outboard motors are not too satisfactory due to difficulty in starting and tendency to drown out in choppy waters (as well as noise). Recommendations resulting from experience with gas engines indicate that light weight diesel engines of higher horsepower ratings are more desirable. Information available at this activity does not disclose the development and use of this equipment to date.

The use of light anchors dropped to seaward of the breaker area has proved advantageous. During tests at Camp Pendleton, California, a light anchor was dropped just seaward of the predominant breaker area and line paid out from the boat as it traversed the surf zone towards the beach. After completing the reconnaissance mission the crew pulled themselves seaward along the cable through breakers of 6 to 8 feet without difficulty. Also,the duration of the sortie was less than half the time required for a crew paddling a boat through the same distance. It is recommended that all boats be fitted with necessary anchoring equipment.

4. Use of Oil to Calm Seas:

It is believed that use of the calming effect of oil slicks on waves in conjunction with rubber boat operations might warrant investigation. It is generally agreed that oil has little effect on waves in the surf area once they have started breaking, but use of oil does tend to prolong the breaking condition. A possible use of oil might occur when onshore winds are strong enough to cause a breaking sea offshore, for its use may prevent the waves from breaking until they reach the surf line proper.



FIGURE VIIE-1 - A RECONNAISSANCE TEAM APPROACHING SHORE



With swimmers dressed and ready, UDT boat (LCP-R) heads for the beach to be investigated.



At a sign from the boat officer (riding forward, next to the camera) the first pair of swimmers goes into the water. The second pair moves from the gunwale to the rubber boat.

FIGURE VIIE-2 - UNDERWATER DEMOLITION TEAM



On the pick-up, the swimmer signals by raising his left arm. Boat (piloted by officer) comes alongside and pick-up man drops rope loop over the arm.



Pick-up man heaves swimmer into rubber boat and gets ready for the next man. These pick-ups were made at about 8 knots.

FIGURE VIIE-3 - UNDERWATER DEMOLITION TEAM

RUBBER BOAT (SEVEN MEN) SURF CASUALTIES. FIRST RECONNAISSANCE COMPANY, HEADQUARTERS BN., FIRST MARINE DIVISION, CAMP PENDLETON, CALIFORNIA, 21 JUNE - 1 JULY 1949.

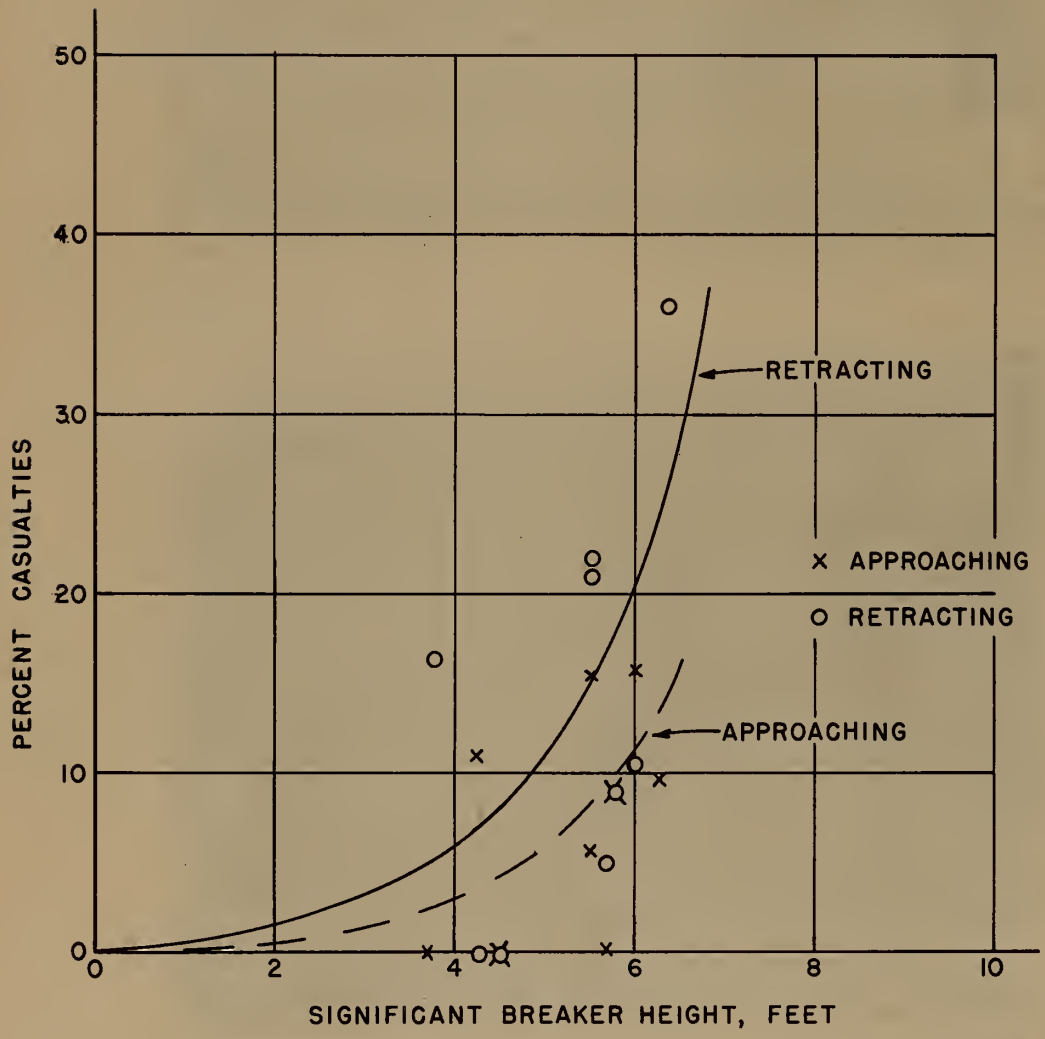





FIGURE VIIE-4

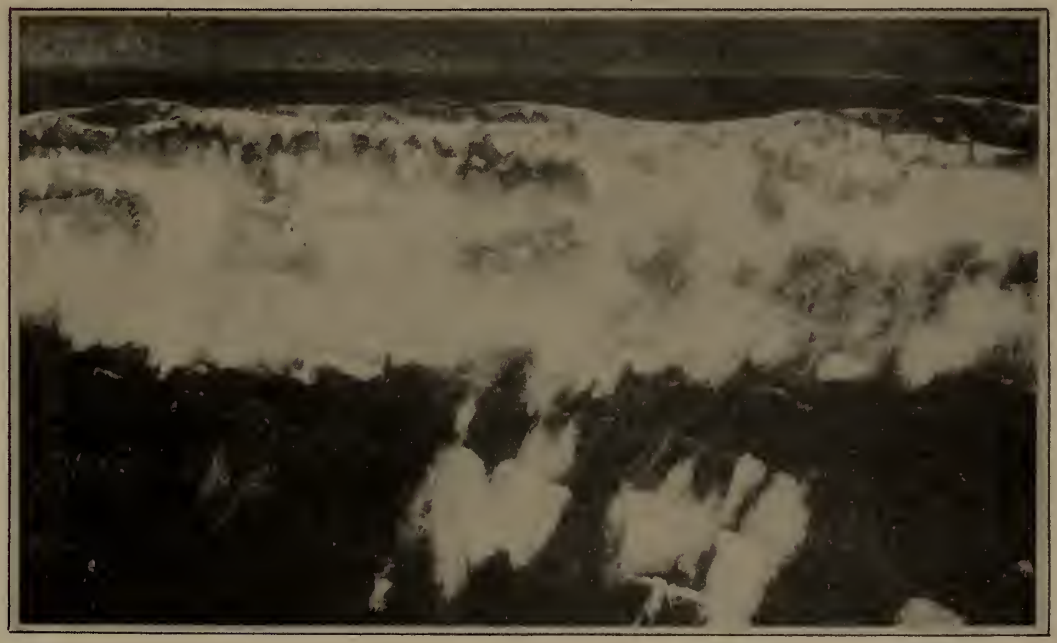
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a. Slope = 1:20. Fort Ord, Monterey, California



b. Slope = 1:50. Santa Margarita River Beach, Oceanside, California



c. Slope = 1:120. Clatsop Spit, Oregon

FIGURE VIIE-5 - EFFECT OF BEACH PROFILE ON WIDTH OF SURF ZONE



FIGURE VIIE-6 - SURF RIPS



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# MANUAL

# of

# AMPHIBIOUS OCEANOGRAPHY

## SECTION VIII

## APPLICATION TO AMPHIBIOUS OPERATIONS

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# MANUAL OF AMPHIBIOUS OCEANOGRAPHY

## SECTION VIII: APPLICATION TO AMPHIBIOUS OCEANOGRAPHY

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C. Ship to Shore Phases

MANUAL OF AMPHIBIOUS OCEANOGRAPHY

SECTION VIII. APPLICATION TO AMPHIBIOUS OPERATIONS

A. TRAINING

by R. L. Wiegel

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## A. TRAINING

### 1. Introduction:

This section does not deal with the preliminary training; that is the training of men to operate craft, drive amphibious tractors, interpret photographs, etc. Rather it will cover the training which is obtained from full scale rehearsals of amphibious landings. It is only with continual training, under a variety of conditions, that men gain skill and confidence. They must work in surf of increasing size and on beaches of increasing difficulty until they are confident and until their seniors are confident that they can successfully perform their missions in combat landings under adverse circumstances. In an operation as complex as an amphibious landing first class training pays off.

Advantage should be taken of every opportunity to obtain information regarding the details of an amphibious operation. What was the surf like in which the craft broached? How skilled was the coxswain? Could he have avoided it? Was the surf forecast correctly? How much in error was the surf forecast? Did the UDT report the proper beach gradient? What was the error? Why? These, and many more questions should be asked, answered, recorded and analyzed. These analyses are of great help in planning future operations.

In war time an amphibious rehearsal should be held on beaches and in surf which closely resemble the actual landing beaches. In peace time they should be held on a variety of beaches under varying surf conditions to develop a versatile command. The operation should be planned so that personnel at all levels will receive training which closely simulates actual combat.

### 2. Intelligence:

Intelligence sections must collect, evaluate and distribute information which is

(i) necessary in the selection of objectives and which is

(ii) necessary in planning and performing the operation. In addition they should observe the operation in order to evaluate the efficiency of the training exercise and to discover methods for improving the efficiency.

As rehearsals are often held at the same location intelligence teams should gain experience by having as their objective other locations than the one specified as the landing site. In fact, intelligence information could be obtained for many locations which could be of future value, while at the same time the personnel are receiving valuable training.

### 3. Forecasting Surf and Beach Conditions:

Operational forecasts should be made as outlined in Part VI of this Manual. There should be observers at the beach (and wave recorders) to record descriptions of the surf at regular intervals so that a check on the accuracy of forecasts can be made. Forecasting is still an art as well as a science and a forecaster of surf has to become acquainted with the "wave climate" of a local area just as he has to become accustomed to the "weather climate" in order to improve his forecasts.

The use of surf forecasts and observations (visual and recorded) in conjunction with known operational limits will be one of the greatest aids to the safety of personnel during training. An aerologist (specially trained in wave forecasting) can usually forecast the surf conditions several days in advance which will be of considerable use in planning training schedules. By keeping an eye on the weather maps any dangerous rise in surf due to local winds can be predicted in enough time to cancel training. Figure VII A-1 shows the rapidity with which the surf can double in height with the accompanying hazard to personnel engaged in training.

4. Specifications for the Oceanographic Coverage of an Amphibious Rehearsal:

In order to obtain certain statistical information on the oceanographic problems of amphibious operations it is necessary to study all scheduled military training operations of this nature. These specifications, based upon previous experience,* outlined the observations that should be made together with the personnel and equipment needed to fulfill the mission.

a. Observations:

The evaluation of the success of a complicated and extensive amphibious rehearsal is a difficult job. The problem of keeping track of exactly what happens at what time and under what conditions are partly solved by the use of aerial and ground observation parties as outlined in this section.

(1) Beach Reconnaissance:

(i) The reconnaissance should be accomplished before the scheduled landings, if possible.

(ii) Set a temporary base line (parallel to the water line) and tie it in with a bench mark, if available. Otherwise, tie it into the tide level. This can be done with fair accuracy (usually within one foot) by considering the limit of the breaker backrush to be the tide level at that time. Readings at the limit of backrush should be made for at least ten waves and the average used.

(iii) Take beach profiles from the permanent back shore features out to where the water depth is about thirty feet below MLLW. It is especially important to locate sand bars or other obstacles, if any.

(iv) The tide cycle should be determined.

(v) Take sand samples at several locations along one of the ranges (perpendicular to the base line). The number of samples to be taken depends upon the variability of the beach. As a minimum selection, a sample should be taken of the berm sand, one at the MLLW line, one at the mid-tide line, one at the MHHW line, and one at sea (in about 20' of water). If the beach material varies along the base line, this should be repeated along several ranges.

* References VIII A-1 through 5.

(vi) Measure and record the direction and magnitude of the littoral current.

(2) Observation Stations:

The number of observation stations will depend upon the length of beach to be subjected to landings and the intensity of the maneuvers.

(3) Aerial Observations During Landings:

(i) Vertical photographs of the landing craft lanes should be made periodically. A clock should be in the camera to give the correct time on each photograph.

(ii) Steep oblique photographs should be made periodically of the landing beaches.

(iii) These photographs should be made so as to best give the distribution of craft with respect to time.

(4) Ground Observations During Landings:

(i) Record time of arrival and departure of craft, together with the type of craft and its number so that the time spent on the beach by each craft may be determined. Also indicate the type of operation such as the "assault", "unloading troops" or "unloading heavy equipment".

(ii) Take photographs from time to time for a permanent record. Be certain to photograph all critical occurances such as craft capsizing or broaching as well as the rescue operations.

(iii) Record wave heights and types, periods and directions from time to time, especially those that cause casualties to either personnel or equipment. Estimate the littoral currents and note the presence of any surf rips.

(iv) Note type of casualties (such as getting stranded on a bar or on the beach--capsizing or being broached), together with surf conditions.

(v) Note trafficability of the beach; i.e. what type of vehicles can run over it and whether they are loaded or not, together with the number of runs that may be made over a particular section of beach before it becomes useless.

(vi) Determine the efficiency of equipment, such as a Pontoon Causeway, in relation to wave height and type, period and direction as well as tide stage.

(vii) Measure tire and track impressions on the beach, together with loading and speed of the vehicles (should be operated at top speed for good correlation) as well as tire pressure if the vehicle has tires. The location from the base line of these measurements should be recorded.

(viii) Take meteorological observations and correlate them with breaker types (plunging or spilling, or percentage of each). This can be done by one of the observation parties.

(5) After Landings:

(i) Question personnel as to the difficulties they encountered while operating both in the surf and on the beach. Also question them about the difficulties in the unloading from supply ships to the amphibious craft.

(ii) Determine the effect of the landings on the beach.

b. Personnel:

The personnel needed will depend upon the number of observation stations that are necessary. There should be one cameraman, one observer and one recorder at each station.

In addition, for the purpose of making the beach reconnaissance, there should be one instrument man, one sounding line man, one recorder, one radio man, and two DUKW drivers. These personnel should later be utilized in the formation of the observation parties as their mission will be accomplished before the landings commence.

5. Example of Coverage and Evaluation:

In conjunction with training it is necessary to discover the reasons for operational failures in order to correct them. The method of covering the effect of oceanographic factors has been outlined in the previous section.

As an example of the coverage of a rehearsal, together with some evaluations consider the training exercise "Operation MIKI" (References VIII A-2 and 5). Representative information obtained from a study of Operation MIKI is presented in this section.

a. Introduction:

The beaches upon which the landings took place were studied both before and after the operation. Meteorological and surf conditions also were studied. Surveys were made on the four beaches which were planned tentatively to be used for the landings (Red, Yellow, Green and Blue). A Wave Recorder was installed in 17½ feet (below MLLW) of water about five hundred feet off the end of the U. S. Army Pier at Pokai Bay. This recorder was in operation during the landings and a continuous record of the waves at Green Beach was obtained.

At 0700 HST on 25 October 1949 (D-Day) observation parties were established at vantage points on the three beaches finally selected for the landings (Yellow, Green and Blue), together with an observation party in an aeroplane. Running accounts were made of the surf and beach conditions, on the craft landings, retractions and failures and on the difficulties encountered by the mobile equipment on the beach. Photographs were taken of these occurrences with the actual time recorded on the picture.

Observed conditions before and during the operation are summarized in the following table. It is assumed that the reconnaissance groups of the invading forces made similar observations as part of the MIKI exercise. These data therefore permit an evaluation of the extent and accuracy of these reconnaissance studies.

TABLE VIII A-1

| Type of Oservation | Observed Data (Refers to Reference VIII A-5) |
|---|---|
| Beach Profiles for Comparison with U.D.T. group, etc. | See Figs. 24 to 29 and Appendix IA |
| Sand Analyses | See Figs. 30 to 33 |
| Beach Trafficability | See Appendix IB |
| Effect of Refraction (on wave heights at various beaches) | See Figure 34 |
| Forecast Wave Heights and Periods | See Appendix II A |
| Observed Heights & Periods During Operation | See Appendix II B and II C |

b. Beach Conditions:

Base lines were established and tied into known features such as bench marks or triangulation stations. These base lines, together with ranges along which the beach profiles (Figure VIII A-2) were taken were located on aerial photographs. In addition to profiles, sand samples and cone penetrometer readings were taken at various locations along the ranges as shown on the profiles of the beaches.

The onshore beach slopes were steep, ranging from 1:4.8 to 1:8.8 while the offshore slopes were gentle with almost all (except one at 1:38 and one at 1:45) ranging between 1:67 and 1:120.

Sieve analyses of the sand samples were made and summarized. Microphotographs and mineral analyses were also made of the beach material.

c. Weather and Surf Conditions:

Observations of swell and surf were commenced on 20 October 1949. Two observations a day were taken, one in the morning and the second in late afternoon. These reports were telephoned to the Fleet Weather Central, Pearl Harbor for their information. They, in turn, reported them to the duty officer, Joint Staff, MIKI, at Fort Shafter. An observation was made at 0000 HST the night of 25 October and hourly observations were started at 0600 HST on 25 October and continued thereafter until 1700 HST on 26 October. Observations on 27 October were made at 0700, 1200 and 1600 HST and again at 0700 HST on 28 October, after which time Joint Staff, MIKI, reported that operations had been secured.

It was found to be very difficult to observe period and direction from any of the landing beaches. An investigation was made of the coast in the vicinity of the beaches, and a point about 4 miles north of Blue Beach was selected for

observing period and direction. The site selected was opposite Makua Cave and situated about 150 feet above the beach, thus affording an unobstructed view of Kaena Point about 8 miles to the north. There were numerous check points for clocking the wave period. Except where noted in the surf observations (Table VIII A-3), all observations of direction and period were made from this point.

Before the landings it was noted that all swell was breaking from the N to the S along the beaches and from the observations at Makua Cave they were observed to be from the NW. On two or three occasions a secondary train of waves, with heights from one to two feet, was observed coming from the southerly quadrant with periods from eight to ten seconds. It was believed that they were caused by the northeasterly trade winds and were being refracted around the islands. Observations reported to the Weather Central from observers on the windward side of Oahu indicated higher surf and swell than normal coming from the northeast.

The weather charts of the North Pacific from October 19 throughout the period during which observations were made (which included the landing dates) showed a series of low pressure systems along the coast of Japan, moving in a general direction of northeast to east. In these systems, winds of Beaufort Scale 6 to 8 (22 to 40 knots), duration of 6 to 30 hours, and fetches of 350 to 1000 miles existed and were found to be the source of the surf that was being observed. Forecasts prepared by the Weather Central (Table VIII A-2) for these conditions indicated that breakers from 2 to 6 feet would exist. Observations (Table VIII A-3) showed that the breakers ranged from 2 to 10 feet, varying with the different beaches. Green Beach was protected from northwest swells by Muana Point and therefore was under the influence of highly refracted waves during the entire time. Blue and Yellow Beaches, being exposed to the northwest, were not protected and therefore the forecast and observed breakers checked within reasonable limits.

A refraction diagram (15 second wave from the NW) was prepared and is shown in Figure VIII A-2.

The onshore slopes of all three beaches were relatively steep (1/4.8 to 1/8.0). Because of this, over ninety percent of the waves broke in a zone from 10 to 30 feet from the beach with a "reflected" wave often being formed as the uprush ran down the beach face. This often caused a peaking up of the oncoming waves above their normal heights. Reflected waves were observed almost continuously.

The type of surf (surging breakers) encountered on the beaches during the landings was typical of long period waves (approximately 16 seconds) encountering steep beaches. Instead of breaking in the normal sense, and expending their energy in so doing, the water would rush up the beach face with a relatively high velocity. This made it difficult for light craft to maintain a proper landing position while unloading troops and equipment.

The local weather during the period of observations was ideal with scattered to broken cloudiness and good to excellent visibility. Surface winds were light to moderate, with a seabreeze coming in, in the late forenoon, nearly every day which increased local "chop" but not, however, materially affecting the breaker heights.

d. Summary of Operations:

Examples of aerial coverage are shown in Figures VIII A-11 through 15.

As an example of ground coverage, the records from observations at Blue Beach will be shown. The first waves of six (6) LCVPs hit the beach at 1003. The seventh LCVP hit the beach at 1005. By 1006 two craft had retracted but four (4) had broached and one (1) was hard aground (Figure VIII A-8A).

At 1008 the second wave of six (6) LCVPs landed (Figure VIII A-8B) and at 1010 one more LCVP hit the beach. By 1011 (Figure VIII A-8D) there were ten (10) craft broached and one (1) hard aground. By 1012 all craft were either hard aground or broached. At 1012½ two (2) LCVPs washed free and retracted.

At 1014 the first LCVP of the third wave landed; at 1014½ four (4) more LCVPs hit the beach; at 1015 one (1) LCM hit the beach and at 1015½ the last LCVP of the third wave landed (Figures VIII A-8E and F). By 1017 one (1) LCVP had lost its crew and one (1) LCVP had retracted. Fourteen (14) craft were either broached or hard aground.

At 1018 the fourth wave turned around and headed out to sea before entering the surf zone.

At 1022 one LCVP had been knocked on its side by a wave and then righted again while another had washed out to sea with its engine out of commission. A salvage craft eventually got a line to it and towed it out to sea by 1832.

At 1025 a salvage craft had shot a line to one of the hard aground LCVPs (PA 233-3). A line was shot to another craft (PA 237-9) which was finally pulled free by 1036½. At 1053 the LCVP (AKA-108-22) was washed off the beach by a large wave and then got under way by its own power. At 1055 the LCVP (AKA-108-11) was pulled off the beach by a salvage crew.

By 1056 one LCVP (PA 237-4) was fairly well broken up by waves pounding on it after its side had been stove in by another LCVP hitting it under wave action. At this time there were nine (9) LCVPs and one (1) LCM hard aground and one (1) LCVP sunk in the surf zone.

At 1110 a Jehemmy was towed onto the beach. It had trouble backing over a craft as the large waves would move them about. However, by 1130 it was finally placed over a LCVP. The Jehemmy was unable to free the craft, but instead both Jehemmy and tractor became bogged down in the sand.

By 1243 two more LCVPs had been pulled off the beach by salvage craft. By this time three (3) LCVPs had been broken up by the surf.

At 1330, when the observation party secured, there were eight (8) LCVPs and (1) LCM either hard aground or broken up.

A summary of arrivals and departures of craft is shown in Figure VIII A-5.

e. Discussion:

(1) Yellow Beach: As on most beaches, the surf height varied along the beach. On October 25 the beach master brought almost all LCMs, LSUs and the LST in at a section where the surf was at the maximum height. A large number of the LCVPs were landed on the exposed rock section of the beach. These two cases emphasize the need of what might be termed "surf and beach judgment" so that craft may be landed on the safest section of a given beach.

In spite of the fact that all wheeled vehicles bogged down as soon as they came off the ramps, no bulldozer was ever held in readiness upon the beaching of craft or ship to quickly aid the wheeled vehicles. Also, in many instances, wheeled vehicles were loaded on the forepart of a ship with the result that these vehicles bogged down in the sand, and the tracked vehicles behind them were prevented from landing or aiding in the recovery of the stuck vehicles.

The LVTs had no difficulty in coming through the surf even though the waves were high. The reason for this was due to the long period of the waves and the beach steepness. The waves, under these circumstances, did not have a steep-front face until they were almost on the beach. At the beach the waves surged up the beach rather than plunging or spilling as happens on less steep beaches. The LVTs surf-boarded a bit but, again because of the steepness of the beach, the tracks did not hit bottom (with the accompanying effect of them turning end-over-end) until the wave lost most of its height to the surge up the beach face. The LVTs then immediately moved on up the beach with no difficulty because of their track laying method of propulsion.

Many occurrences indicated the lack of judgment on the part of operators. A particular instance was the towing through the surf of a stalled LVT by another LVT. When the towing LVT had driven onto the beach, the line was allowed to become slack with the immediate effect of it falling off of the following craft. The disabled LVT, still in the surf, was exposed to the full effect of the waves and immediately became broached.

A most interesting occurrence at this beach showing the power of waves can be seen in Figure VIII A-6. An LCM, approaching the beach, was pushed onto the beach by a wave and partially broached. Another wave then turned the craft broadside while the next wave turned it completely around and the backrush started to carry it out to sea again. The coxswain immediately put it on full throttle and headed back through the surf and out to sea, never having unloaded his craft.

(2) Green Beach: The surf on Green Beach was relatively low, varying between two and three feet at the beach extremities to from one to two feet in the center. There was, therefore, an almost negligible surf problem.

Throughout the entire time the observation party was taking notes on the first day (25 October 1949) there were only a few craft that got into serious difficulties. The first, an LCVP, broached because of the confusion existing while the coxswain and a member of the Beachmaster's staff were shouting back and forth. It was difficult enough for the coxswain to land properly without being interrupted by a member of the beach party who in turn was actually confused. Under these circumstances the coxswain attempted to retract with the ramp down and the assault personnel still aboard with the result that the craft swamped and then broached.

An attempt to salvage this craft by the aid of a Jeheemy was made. It was not successful because of the difficulty the Jeheemy had in the soft sand. Instead, the Jeheemy had part of its frame badly bent. The LCVP became a complete loss. There also was a LCP(S) hard aground on GreenBeach. The reason for its broaching was not determined. It was later returned to sea by a Jeheemy.

During the late afternoon (about 1800) another LCVP was swamped. There was a hole in the stern of this craft. The coxswain had forewarned the supply ship before this landing was made, but nevertheless it was sent into the beach with the cargo concentrated aft. The craft became a total loss.

The docking of LSTs were sometimes very slow. On one occasion, on 26 October, it required two and a half hours for the ship to dock at the end of the Pontoon Causeway.

The main difficulty at this beach was in the unloading of wheeled vehicles. All of them, with the exception of several of the DUKWs which had deflated their tires, bogged down. Even after wire matting had been placed, several vehicles got into trouble. Several DUKWs bogged down because their drivers failed to deflate their tires. The sand was so "soft" that even some tracked vehicles encountered difficulty.

On the second day (26 October) all unloadings were concentrated on about a one hundred yard section at the south end of the beach where the only vehicle exits from the beach were situated. Mats were laid on the sand and bulldozers stationed to pull vehicles up the steep embankment. The traffic was very heavy at this section.

The main casualty of the day was due to misjudgment on the part of the coxswain. The craft was at "idle" when the ramp was lowered, with the result that the first small surge moved it a small distance out from the beach allowing water to pour in through the bow.

Several other craft started to broach, and were pulled free immediately by salvage craft. The operators of some of the salvage craft were relatively slow to act in many instances. If any surf had existed on this beach these craft probably would not have been salvaged.

(3) Blue Beach: The surf averaged about five feet. At about five (5) to ten (10) minute intervals a series of four (4) to eight (8) large waves (from 8 to 10 feet) would arrive. The beach face was very steep and the sand afforded poor trafficability. There were large cusps in the beach. The waves, instead of breaking, would usually peak up as if to break and then transfer into a translational motion--surging up the steep beach face. Thus, the wave energy, instead of largely being dissipated by the breaking process, was available to rush up the beach, thus pushing the craft far up onto the beach. At the same time the waves lifted the LCVPs up a little and then dropped them hard on the beach, driving the rudder into the sand. Thus, the coxswain lost all control of the craft and the next few large waves completely broached the craft. When the next group of large waves arrived, the craft were washed even farther up the beach.

On some occasions it was impossible to raise the ramps. Consequently the craft filled with water and sank.

Salvage operations were hampered by the rapidity with which the craft got part of their hulls buried in the soft sand.

Too much time was spent in lowering ramps, getting the troops out and raising ramps again. On the beach such as this, it is particularly important to retract as soon as possible. Secondly, the salvage crews showed little judgment. Instead of throwing a line to a craft that was not hard aground too badly, they threw lines to the craft which were in the most difficult positions with very little chance of being pulled free.

f. Conclusions:

(i) All the beaches encountered on Operation MIKI, Hawaiian phase, had steep on-shore slopes (1:4.8 to 1:8.8) and "soft" sand. Thus the beaches were very undesirable from a trafficability standpoint. All wheeled vehicles, with the exception of the DUKWs which deflated their tires, became stuck in the sand. In addition tracked vehicles became stuck on occasions when they were under a heavy draw-bay load.

(ii) Because of refraction, the waves at Green Beach were low (1-3 feet) but higher at Yellow Beach (3-8 feet) and the highest (4-10 feet) at Blue Beach. Because of the long periods (16 seconds) and the steep beach faces the waves surged up the beach rather than plunging or spilling in a true "surf zone".

(iii) The surf and beach conditions at Yellow and Blue Beaches proved hazardous to light landing craft such as LCVPs and LCMs. This was because almost the entire energy of the waves, instead of being largely dissipated in breaking, changed into a surge up the beach at the very point that the landing craft was to unload its cargo. Thus the craft were literally pushed up onto the beach and then stranded.

(iv) The surf and beach conditions at Yellow and Blue Beach were not difficult for LVTs. The main hazard exists because the waves surge up the beach rather than break. Thus, the LVT, which continues to move up the beach is barely affected. Also, being a track laying vehicle, it had no difficulty moving over the beach.

(v) Many craft spent an excessive length of time on the beach because of ramp difficulties. A landing craft must get off the beach as soon as possible and one of the main reasons for slowness in retraction was due to ramp trouble. Either a new type of ramp motivation should be utilized or the present ramps made to operate more effectively.

(vi) The operators of salvage craft often showed poor judgment in throwing tow lines to craft in the greatest difficulty rather than to ones which, with quick help, could be rescued. There were many instances where more rapid response by salvage craft personnel could have proved beneficial.

(vii) On Yellow Beach, and especially on Blue Beach, it is doubtful whether any but the most skilled coxswains could have improved the number of retractions. On Green Beach, however, almost all difficulties were due to poor surfmanship on the part of the coxswains.

(viii) The Beachmaster at Yellow Beach showed poor "surfmanship" in directing landings to be made in the section of highest surf.

g. Tables:

(1) VIII A-2 Surf Forecasts by Commander T. D. Keegan, USN, Fleet Weather Central

| | | |
|---|---|---|
| Saturday | - 22 October - | Average 3' - 4' breakers on beaches. |
| Sunday | - 23 October - | Average 2' - 3' breakers on beaches becoming 3' - 5' in evening. |
| Monday | - 24 October - | Average 3' - 5' breakers on beaches. |
| Tuesday | - 25 October - | Average 3' - 4' breakers on beaches. |
| Wednesday | - 26 October - | Blue Beach Average 5' Maximum 7' - 8'<br>Green Beach Average 2' Maximum 4'<br>Yellow Beach Average 4' Maximum 6' |
| Thursday | - 27 October - | (Breaker heights increase about noon)<br>Blue Beach Average 5' Maximum 8' - 10'<br>Green Beach Average 2' - 3' Maximum 4'<br>Yellow Beach Average 5' Maximum 7' - 9' |
| Friday | - 28 October - | Blue Beach Average 5' Maximum 7'<br>Green Beach Average 2' - 3' Maximum 4'<br>Yellow Beach Average 4' Maximum 6' |

NOTE: 1. For breakers on North and South end of each beach add 1' - 2' to average above.

2. Breakers for area between Kaena Point and Kapuhi Point average 3' - 4' above beach breakers.

3. Occasional 7' - 10' breakers noticed Saturday, 22 October on southern two thirds of Blue Beach only. This is a local condition due apparently to refraction of the northerly swells off MAUNA LAHILAHI. This is expected to continue through Sunday.

4. Towards noon each day a local sea breeze effect near beaches will increase choppiness but not materially effect breaker heights.

(2) VIII A-3 Surf Observations, Sample

| Date Oct. | Location | Time HST | Period, Seconds | (H)1/3, Ft. | $H_{max}$, Ft. | Direction | Remarks |
|---|---|---|---|---|---|---|---|
| 24 | Blue | 1440 | | | 8-10 | | 30-40% of all waves br. at that height. |
| 24 | Green | 1500 | | 1½ | 3 | | |
| 25 | Blue | 0000 | | 6 | 8-10 | | |
| 25 | Green | 0000 | | 2 | 3 | | ½' to 1' lower in center |
| 25 | Blue | 0545 | 16.5-17.0 | 6 | 8-10 | | Berm dev. on N end since Midn. |
| 25 | Green | 0555 | | 2 | 3 | | ½' to 1' lower in center |
| 25 | Yellow | 0620 | | 3-4 | 5 | | |
| 25 | Green | 0630 | | 2 | 3 | | |
| 25 | Blue | 0740 | | 6 | 8-10 | | Wave surging to upp. limits of berm |
| 25 | Green | 0750 | | 2 | 3 | | |
| 25 | Blue | 0840 | 17.0 | 6 | 8-10 | | |
| 25 | Green | 0850 | | 2 | 3 | | ½' to 1' lower in center |
| 25 | Blue | 0940 | | 4-5 | 7-8 | NW'ly | 5-10% |
| 25 | Green | 0950 | | 2 | 3 | | ½' to 1' lower in center |
| 25 | Yellow | 0920 | | 3-4 | 5-6 | NW'ly | |
| 25 | Yellow | 1015 | | 4-5 | 6-7 | | |
| 25 | Blue | 1040 | 16.5-17.0 | 5-6 | 8-10 | NW'ly | 10-20% |
| 25 | Green | 1050 | | 2 | 3 | | ½' to 1' lower in center |
| 25 | Yellow | 1115 | | 5-6 | 7-9 | | |
| 25 | Blue | 1140 | 16.5-17.0 | 6 | 8-10 | | 10-20% of all waves br. at that height |
| 25 | Green | 1150 | | 2 | 3 | | ½' to 1' lower in center |
| 25 | Yellow | 1215 | | 5-6 | 7-9 | | 10-20% |
| 25 | Blue | 1240 | | 6 | 8-10 | | 10-20% |
| 25 | Green | 1250 | | 2 | 3 | | ½' to 1' lower in center |
| 25 | Yellow | 1315 | | 5½-6 | 7-9 | | |
| 25 | Blue | 1340 | | 5½-6 | 7-9 | | |
| 25 | Green | 1350 | | 2½-3½ | | | ½' to 1' lower in center |
| 25 | Yellow | 1415 | | 5-5½ | 7-8 | | |
| 25 | Blue | 1440 | | 5½-6 | 7-8 | | |
| 25 | Green | 1450 | | 3 | 3½ | | ½' to 1' lower in center |
| 25 | Yellow | 1515 | | 5-5½ | 7-8 | | |
| 25 | Blue | 1540 | | 5½-6 | 7-8 | | |
| 25 | Green | 1550 | | 3 | 3½ | | ½' to 1' lower in center |

(3) VIII A-4 Wave Records From Mark V No. 3
Pokai Bay, Hawaii T.H. October 22-26, 1949

| Date | Time (HST) | Period (Seconds) | Significant Breaker Ht. (Feet) | Maximum Breaker Ht. (Feet) |
|---|---|---|---|---|
| 22 | 1450-1511 | 12.8 | 1.7 | 2.6 |
| 23 | 1100-1120 | 12.1 | 1.2 | 1.9 |
| 24 | 0945-1005 | 14.3 | 1.3 | 1.9 |
| 24 | 1501-1523 | 15.2 | 2.1 | 3.1 |
| 25 | 0752-0812 | 15.8 | 2.1 | 3.2 |
| 25 | 0852-0912 | 15.6 | 1.7 | 2.2 |
| 25 | 0952-1012 | 15.8 | 2.1 | 3.0 |
| 25 | 1052-1112 | 15.4 | 1.9 | 2.7 |
| 25 | 1152-1212 | 15.2 | 2.2 | 2.8 |
| 25 | 1252-1312 | 15.8 | 2.0 | 2.4 |
| 25 | 1439-1515 | 15.0 | 2.7 | 4.0 |
| 25 | 1550-1610 | 14.6 | 2.6 | 3.7 |
| 25 | 1650-1710 | 15.2 | 2.6 | 3.8 |
| 25 | 1750-1810 | 14.6 | 2.2 | 3.6 |
| 25 | 1850-1910 | 14.3 | 1.7 | 3.0 |
| 26 | 0822-0842 | 12.1 | 1.2 | 2.7 |
| 26 | 0954-1008 | 12.3 | 1.0 | 1.5 |
| 26 | 1129-1144 | 12.3 | 1.1 | 1.5 |
| 26 | 1303-1320 | 12.6 | 1.1 | 1.5 |
| 26 | 1515-1531 | 12.2 | 1.1 | 2.0 |

6. References:

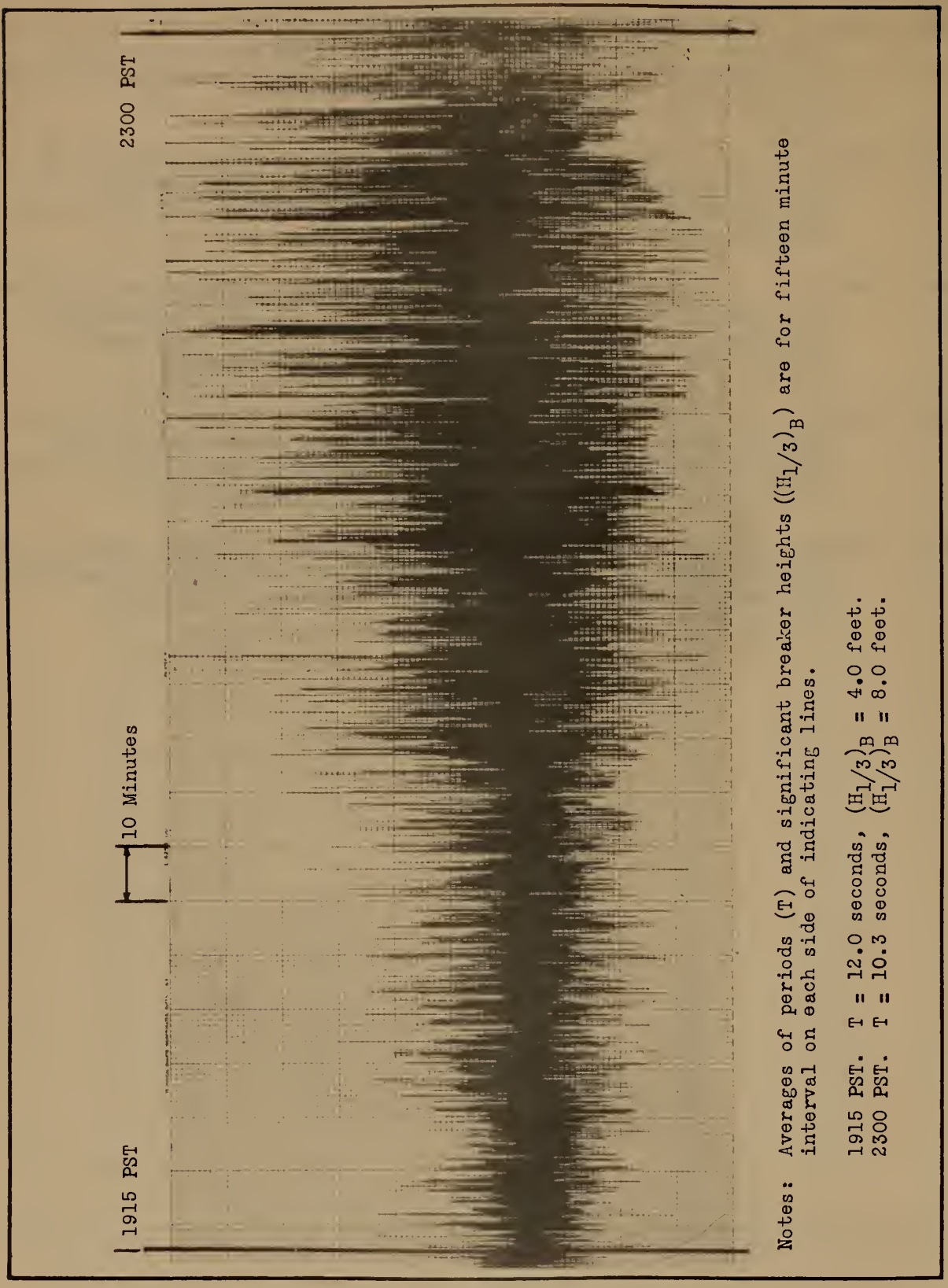
VIII A-1: Bascom, W. N., Wiegel, R. L. and Chinn, A. J. - "Oceanside Amphibious Operation, November 1947" - Series 3, Issue 293, Institute of Engineering Research, University of California, Berkeley, California, November 1948, (unpublished).

VIII A-2: Bascom, W. N. - "Operation MIKI, Hawaiian Phase (Aerial Coverage)" - Series 29, Issue 15, Institute of Engineering Research, University of California, Berkeley, California, 25 October 1949, (unpublished).

VIII A-3: Isaacs, J. D., Bascom, W. N. and Foight, D. F. - "Report on Amphibious Operation at Oceanside, November 1948" - Series 3, Issue 325, Institute of Engineering Research, University of California, Berkeley, California, 29 January 1947, (unpublished)

VIII A-4: Patrick, LTJG D.A. - "Operation MIKI, Rehearsal at Oceanside (Lex 4), California on 9 October 1949 (Ground Coverage)" - Series 29, Issue 13, Institute of Engineering Research, University of California, Berkeley, California, 15 February 1950, (unpublished).

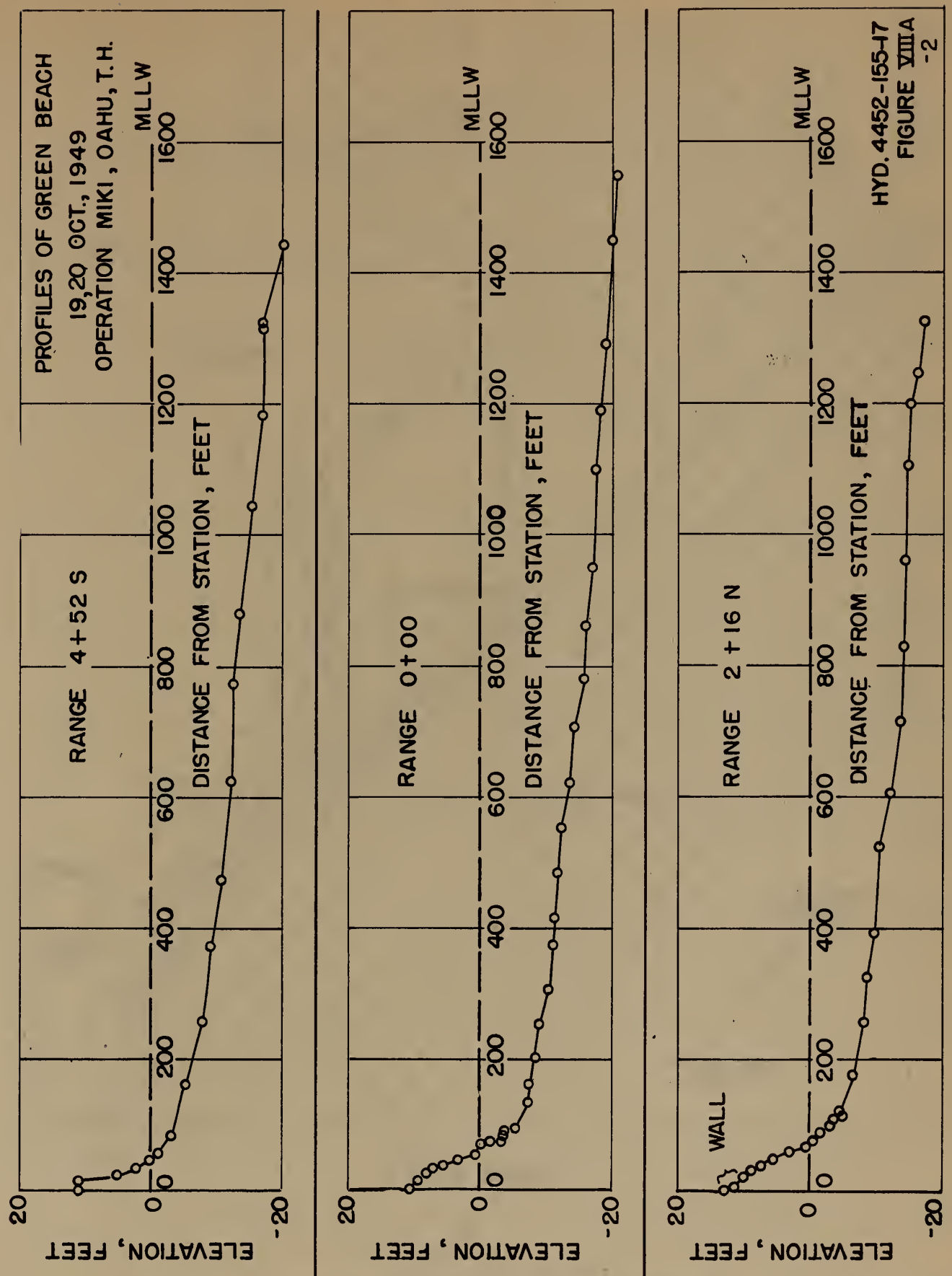
VIII A-5: Wiegel, R. L. and Kimberley, Lt. H. L. - "Operation MIKI, Hawaiian Phase (Ground Coverage)" - Series 29, Issue 14, Institute of Engineering Research, University of California, Berkeley, California, 25 October 1950, (unpublished).




Notes: Averages of periods (T) and significant breaker heights ($(H_{1/3})_B$) are for fifteen minute interval on each side of indicating lines.

1915 PST. T = 12.0 seconds, $(H_{1/3})_B$ = 4.0 feet.
2300 PST. T = 10.3 seconds, $(H_{1/3})_B$ = 8.0 feet.

FIGURE VIII A-1 - Mark V #1A record for 1915 to 2300 PST, 10/18/49, Camp Pendleton, Oceanside, California


PROFILES OF GREEN BEACH
19,20 OCT., 1949
OPERATION MIKI, OAHU, T.H.
MLLW
RANGE 4+52 S
RANGE 0+00
RANGE 2+16 N
DISTANCE FROM STATION, FEET
0
200
400
600
800
1000
1200
1400
1600
ELEVATION, FEET
20
0
-20
WALL
HYD. 4452-155-17
FIGURE VIIIA
-2

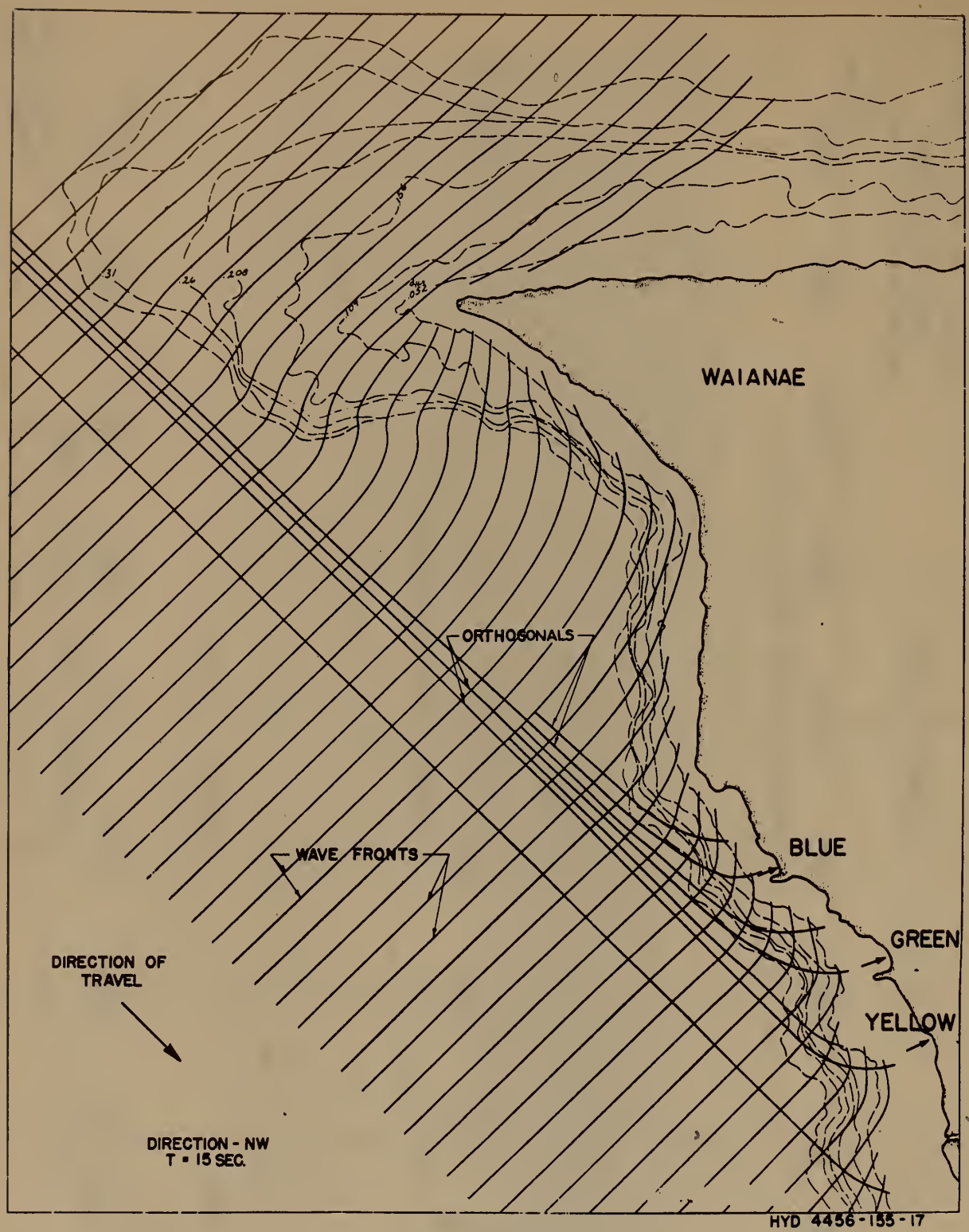
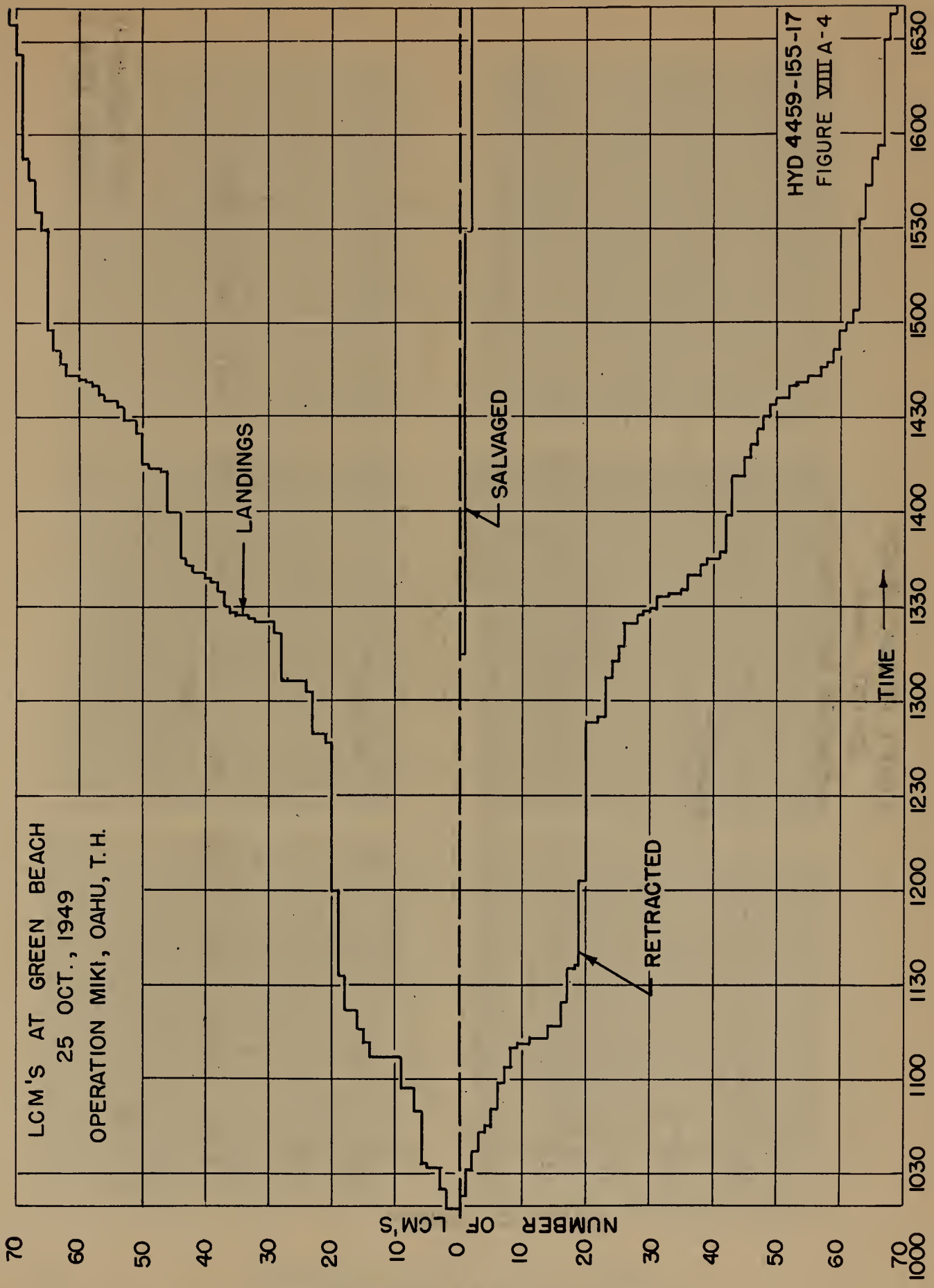
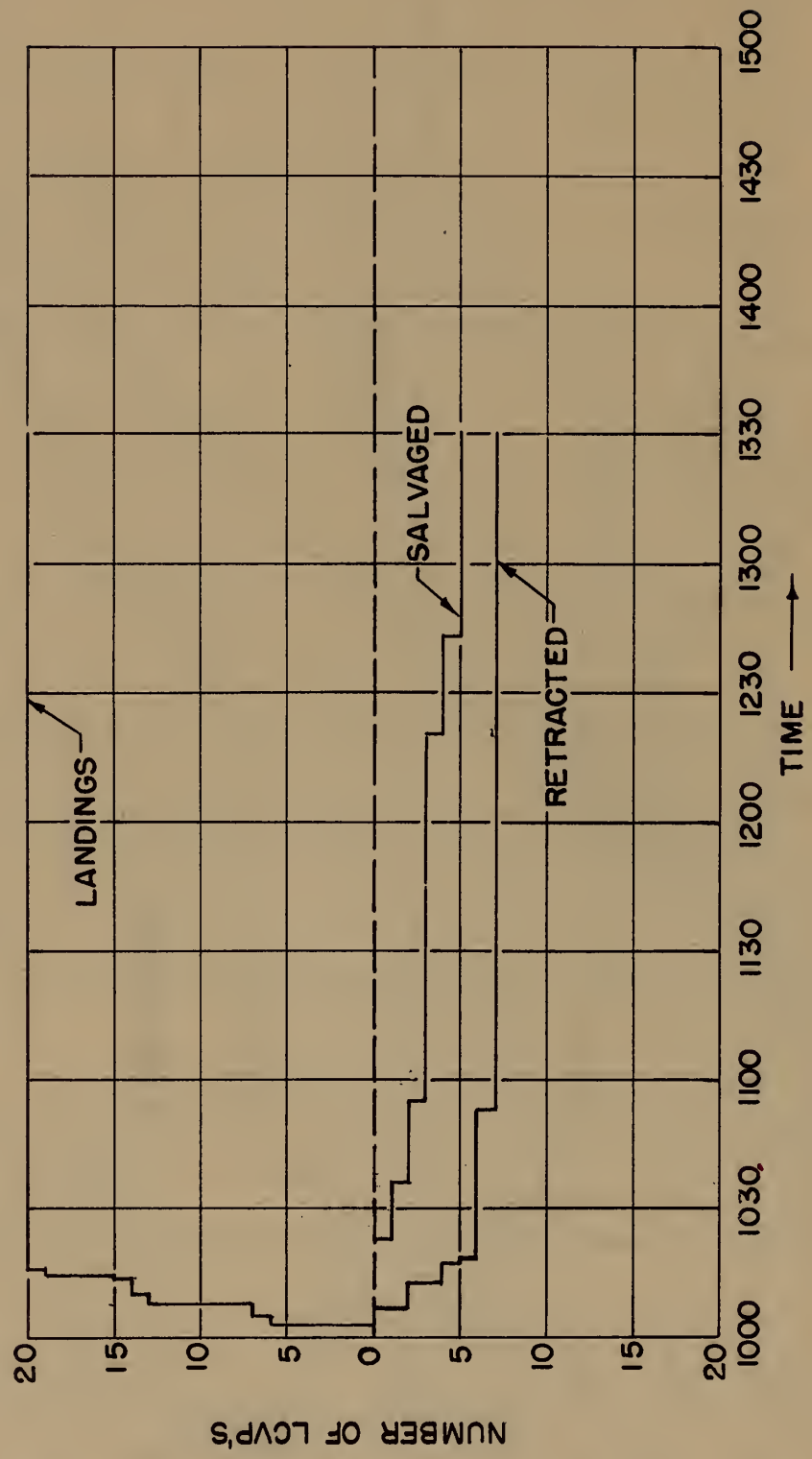



FIGURE VIII A-3


HYD 4459-155-17
FIGURE VIII A-4
LCM'S AT GREEN BEACH
25 OCT., 1949
OPERATION MIKI, OAHU, T.H.
LANDINGS
SALVAGED
RETRACTED
TIME
NUMBER OF LCM'S
70 60 50 40 30 20 10 0 10 20 30 40 50 60 70
1000 1030 1100 1130 1200 1230 1300 1330 1400 1430 1500 1530 1600 1630

HYD. 4460-155-17
FIGURE VIII A-5

LCVP'S AT BLUE BEACH
25 OCT., 1949
OPERATION MIKI, OAHU, T.H.

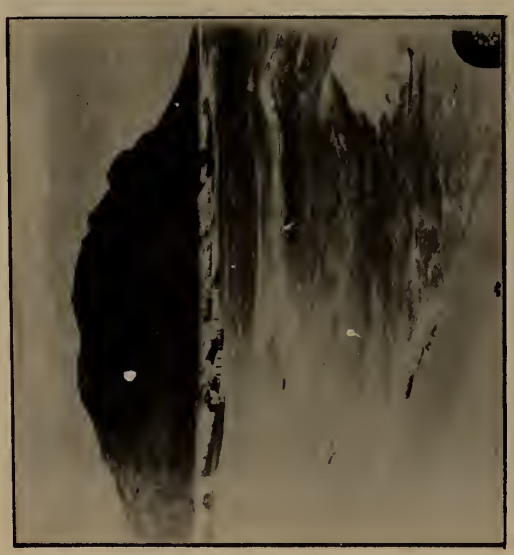




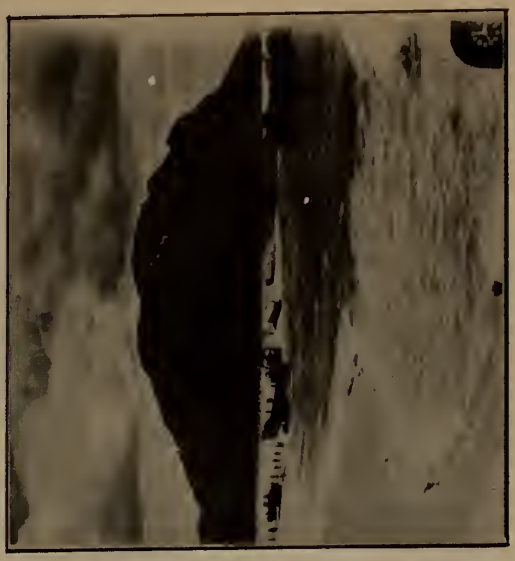
c. 1455 - LSU 1330 surf-boording in on o wave.



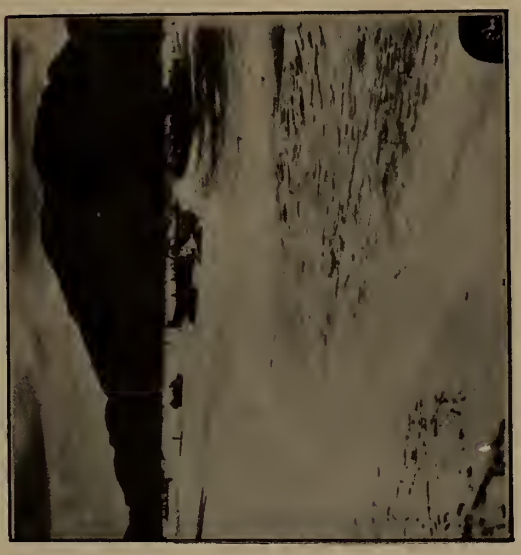
b. 1414 - LSU 1459 londing.



a 1409 - Bulldozer coming to rescue of onother one stuck in surf



f. 1459 - LSU 1330 retracting. Agressor plane attacking beach.



e. 1457 - Vehicles unlooding from LSU 1330.



d. 1456 - Wave pounding LSU 1330.

FIGURE VIII A-6 - YELLOW BEACH, 25 OCTOBER 1949, 1409 TO 1459 HST



a. 0809 - Surf before landings.

b. 1003 - First wave about to land.

c. 1006 - Part of first wave retracting. Remaining craft slow as troops unloading over side due to inoperation of ramps.



d 1009 - Second wave starting to unload.

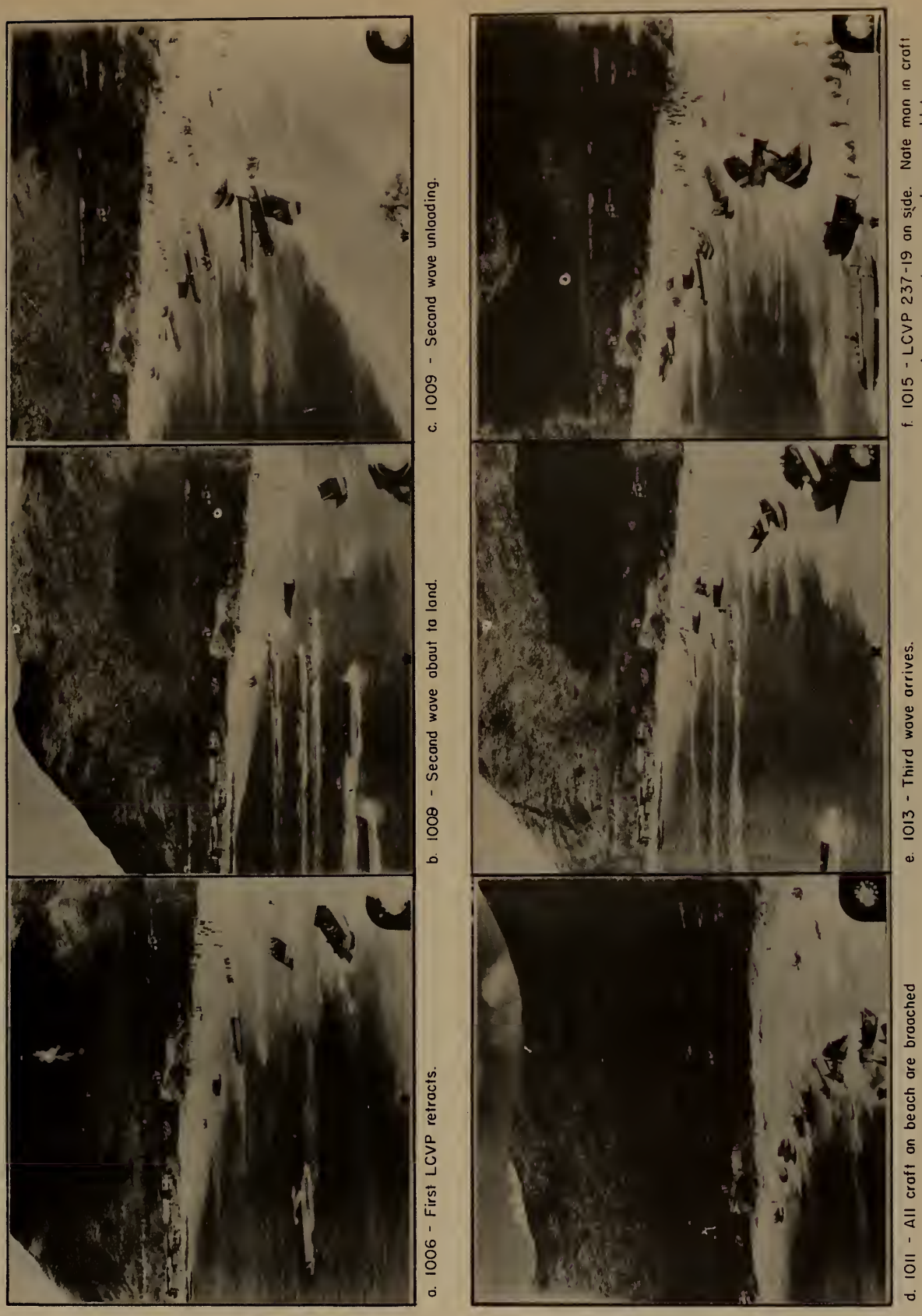
e. 1014 - Only one broached LCVP on beach.

f. 1015 - Third wave starts to unload.

FIGURE VIIIA-7 - GREEN BEACH, 25 OCTOBER 1949, FIRST THREE WAVES



RESTRICTED Security Information



a. 1006 - First LCVP retracts.
b. 1008 - Second wave about to land.
c. 1009 - Second wave unloading.
d. 1011 - All craft on beach are broached
e. 1013 - Third wave arrives.
f. 1015 - LCVP 237-19 on side. Note man in craft Aggressor plane spreading pamphlets

FIGURE XIII A-8 - BLUE BEACH, 25 OCTOBER 1949, SECOND & THIRD WAVES



b. Green Beach, cargo section.



d. UDT marks sunken craft location.



a. Congested beach.



c. Pumping water out of an LCVP.

FIGURE VIII A-9 - GENERAL SCENES, 26 OCTOBER 1949



b. Stuck truck.



d. Stuck dukw.



a. Stuck jeep.



c. Stuck tank retriever.

FIGURE VIII A-10 - SOFT SAND, STEEP BEACH

a. 0920, 10/26/49 - Disposition of ships off the Waianae Pocket. All landing activity is centered on Green Beach.



FIGURE VIII A-11



b. 0925, 10/26/49 Transport area and craft patterns seen from the north. (These two photos of the same group of ships show the great foreshortening effect of the 24-inch lens.

K-18 photos



a. 1047 - Green Beach; one poorly-defined wave of 8 boats approaching the beach through 11 retracting craft.



b. 1351 - 10 loaded LCMs floating about in Pokai Bay, waiting for instructions, although there seems to be ample beaching space. Salvage of a boat by a Jeheemy at the left is underway. There is some evidence (broached boat, screw eddies) of a southerly current on shore at this time.



c. 0930, D+1 - Green Beach on the second day had few boats beached but many floating aimlessly about in the boat lanes.



d. 0941, D+1 - LST disgorging onto a pontoon causeway. Transports have moved in close. LCT is bravely plowing through a maze of hove to craft. Two sunken boats on the beach have apparently been abondoned.

F-56 photos

FIGURE VIIIA-12 - GREEN BEACH, OAHU, T. H., 25-26 OCTOBER 1949



RESTRICTED Security Information

a. 1016 - Blue Beach; 15 boats broached and/or hard aground A salvage boat is working in close (in danger of catching a breaker, or surfboarding in). 4 of the broached boats still have their romps down.

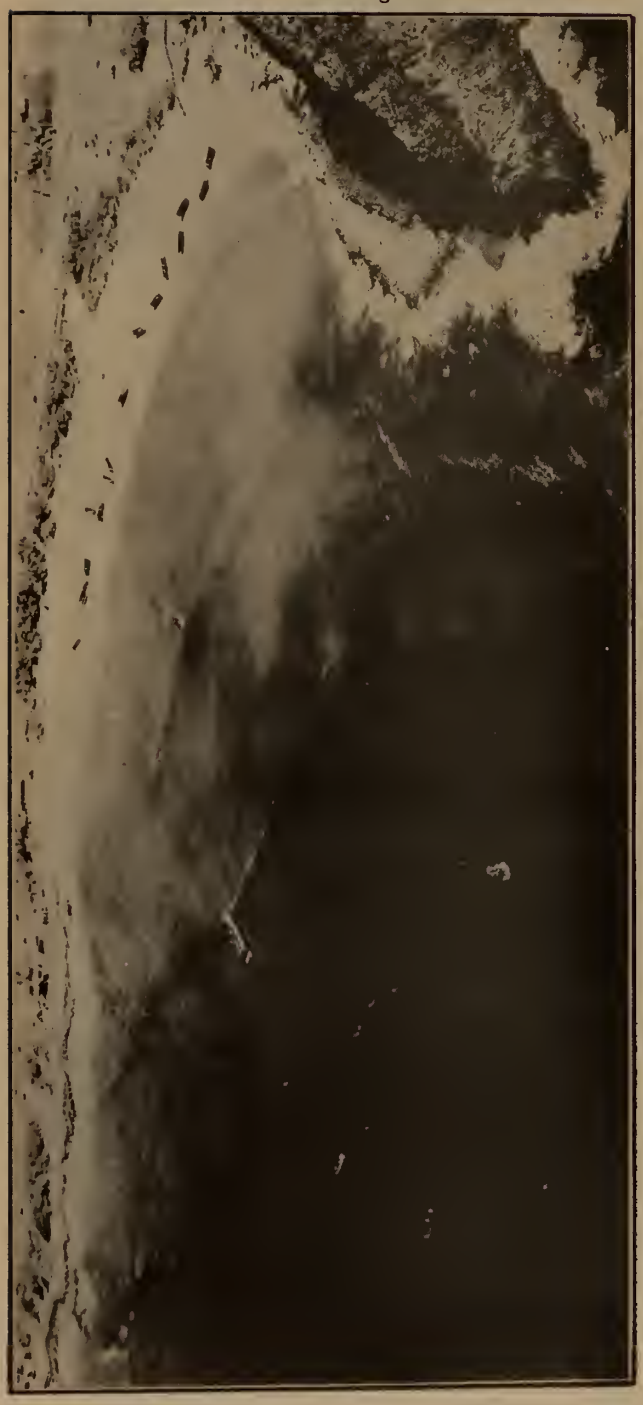


FIGURE VIII A-13
WAIANAE, OAHU, T.H.,
25 OCTOBER 1949



b. 1020 - Green Beach, fifth wave approaching, fourth wave retracting

K-18 photos

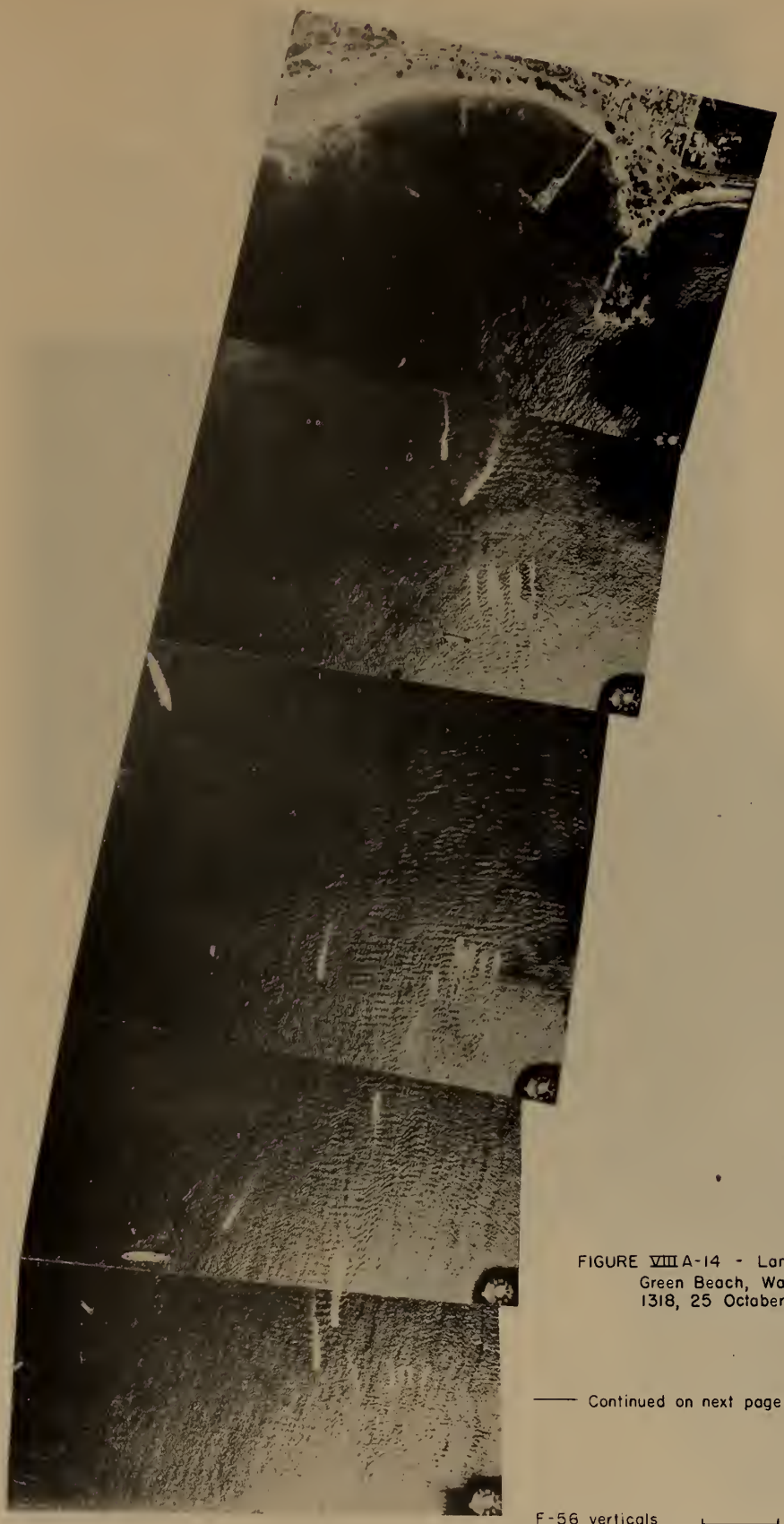


FIGURE VIII A-14 - Landing craft lane, Green Beach, Waianae, 1318, 25 October 1949

—— Continued on next page

F-56 verticals ⊢——⊣



Continued from previous page

FIGURE VIII A-15 - Landing craft lane
Green Beach, Waianae,
1318, 25 October 1949

F-56 verticals

0' 500'

SECTION VIII. APPLICATION TO AMPHIBIOUS OPERATION

## B. OCEANOGRAPHIC INFORMATION NECESSARY FOR EVALUATING SITES FOR AMPHIBIOUS OPERATION

BY R. L. WIEGEL

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## B. OCEANOGRAPHIC INFORMATION NECESSARY FOR EVALUATING SITES FOR AMPHIBIOUS OPERATIONS

1. General Discussion:

In general it is necessary to know the following oceanographic information in order to properly evaluate sites for amphibious landings:

(i) Relationship between beach and surrounding terrain

(ii) Dimensions of the beach

(iii) Configuration of foreshore and offshore sections of the beach

(iv) Shape and orientation of the beach

(v) Exposure of the beach to waves

(vi) Egressability

(vii) Offshore wave climate and surf climate

(viii) Hydrography

(ix) Beach trafficability

(x) Tides

(xi) Fleet hazards

(xii) Holding ground

Because of these many factors and their varying - often conflicting- effects on landings, it is not possible to set down a list of rules for determining which of several locations is the best. The amphibious commander must weigh the various conflicting factors, assess the capabilities of his personnel and equipment as well as the enemy's; then make his decision.

a. Relationship between the Beach and Surrounding Terrain:

Information on this subject has been presented, briefly, in Section I-I of this Manual. It is a complex subject and is more properly covered in manuals on military geology.

Regarding the terrain surrounding landing beaches, the U. S. Marine Corps, Amphibious Manual, AM-2(Terrain, Hydrography, and Weather, 1951) states:

(1) "Terrain Appreciation: Terrain appreciation is the evaluation of the area of probable military operations to determine the effect of the terrain on the lines of action open to opposing forces in the area. The principles of terrain appreciation which are applicable for land warfare are relevant to the amphibious assault. A disregard for or an erroneous application of the principles may result in failure for an otherwise well planned operation. The conditions of terrain must be evaluated with respect to the techniques, equipment, and tactics of amphibious warfare."

(2) "Strategic Considerations: Strategically important terrain considerations are dependent upon the nature of the objective. Thus, flat ground and unobstructed approaches are required for an air base; wheras sheltered anchorages, good holding ground, and freedom from navigational hazards are necessary to a naval base. Usually, a base must have facilities for both naval and air operations, and the island or portion of the coast of a land mass which is under consideration must have the terrain characteristics required for both.

However, no matter what may be the reason for seizure of an island or coastal area, the amphibious operation will be directed toward a definite objective in that area, and the terrain will be of considerable importance in the selection and definition of the target area. Such an area must include landing sites from which a relatively unobstructed (by terrain) movement inland to the objective may be made. Consequently, the landing area should be so selected that mountain ranges of great size, extensive deserts, jungles, or marshes do not block the route of advance or interfere with the over-all conduct of the campaign.

Should the strategic objective of an amphibious operation be merely the seizure of an island or portion of a coast (possibly to deny its use to the enemy), an area in which the terrain provides the maximum defensibility should be selected."

(3) Emphasized Importance of Terrain Characteristics in Amphibious Operations: "When the choice of the landing area has been made, the exploitation of the terrain must be complete. The discovery, analysis and evaluation, and application of every particular of the terrain is of especial importance in amphibious operations because:

(i) A landing operation involves a transition from seaborne to land attack in an area which is in close proximity to of dominated by the defender. The purely mechanical aspects of the transition are complex; when they are conducted under the guns of the enemy, they are crucial. This transition, under extremely vulnerable conditions, from strength of zero on the beach to that required to overcome enemy resistance, requires that there be available space, cover and firm ground. A failure to properly evaluate this area, over which the landing is to take place and in which the build-up of the attack occurs, may result in a disaster of greater magnitude than would occur by reason of a poor local terrain evaluation in land warfare.

(ii) The complexity and precise coordination of an amphibious operation make it essential that the landing be carried to completion after it has commenced. A retreat can only mean a complete defeat. Even delays and postponements usually will have serious consequences. As a rule, only one chance for success is offered. The evaluation of the terrain must be right the first time.

(iii) Prior to the actual assault, subordinate units are not in contact with the enemy, nor have they had any physical acquaintance with the ground over which

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they must operate. Inasmuch as subordinate units have no means for physical reconnaissance of the landing area, a very close study of the terrain must be made by the highest echelon of the force. Prior to H-hour, these detailed terrain studies provide the basis for the plans of every assault unit.

(iv) In land warfare, terrain which is selected for a defensive position usually does not favor the attacking force; and conversely, terrain which is favorable for an attack is difficult to defend. Usually, because of the configuration of the shorelines, an area which is most suitable for the execution of a landing operation is also most easily organized for defense. Thus, the landing force has a doubly difficult task in the analysis of the terrain of the landing area."

b. Beach Dimensions:

The dimensions of beaches under consideration and their importance to the efficiency of an amphibious operation should be emphasized. The beach is the most important section of the beach head and the establishment of a secure and effective beachhead is the initial mission of a landing force. To quote*:

"Without this nothing else is possible. The three major considerations in the determination of a beachhead are:

(i) Sufficient space to accomodate the troops, equipment, and installation of the landing force without dangerously crowding;

(ii) Freedom from enemy small arms fire and ground observed artillery fire; and

(iii) Defensibility."

The necessary size of a beach is largely dependent upon its configuration and upon the surf. Where the beach is mild and the surf is low, more craft can land and discharge their cargo in a given time than on a steep beach or a flat beach with the same offshore waves.

c. Foreshore and Offshore Beach Configuration:

Trafficability of beaches is largely dependent upon the foreshore beach configuration. For example, steep beaches almost always afford poor trafficability while flat sand beaches almost always afford good trafficability.

Surf and beach configuration are interdependent. For example, "steep" storm waves create offshore bars; the bars in turn affect the surf characteristics. Flat beaches usually have several bars, with waves breaking on all of the bars at different tide stages. Moderate beaches usually have but one line of breakers. Steep beaches usually have the waves breaking at the beach face. The effect of beach configurability on surf and the joint effect on different types of landing craft and amphibian vehicles are very crucial in an amphibious operation and have been covered in great detail in Section VII of this Manual (Surf Characteristics and Surfmanship).



* U. S. Marine Corps, Amphibious Manual, AM-2, 1951

d. Shape and Orientation of Beach:

The shape and orientation of beaches largely control the surf characteristics, beach trafficability and general accessibility as well as the exposure of the beach to enemy fire. The complex relationships between beach shape, orientation and wave climate are thoroughly discussed in Section I-I of this Manual (Beaches).

e. Exposure of Beach to Waves:

The exposure of a beach to either prevalent or occasional wave conditions must be given consideration. Normally, landings should not be made on sections of coastline exposed to the prevalent waves; however, during certain seasons of the year, these beaches may be safe. On the other hand, some beaches may be well protected from the prevalent waves and yet be subjected to heavy surf during certain seasons.

The exposure of a beach may be determined by use of hindcasts from weather maps, combined with refraction studies, or by study of large numbers of aerial photographs.

f. Egressability:

The following is quoted from U. S. Marine Corps Amphibious Manual, No. AM-2, 1951:

> "A landing area should be chosen which will allow the force making the landing, or a subsequent force, to move out of the area. The force should not be required to advance through comparatively impassable terrain which is suitable for strong enemy defensive positions. In this respect, along many mountainous shores there exist good beaches which are backed by coastal plains of sufficient size to permit the landing of a comparatively large force; but beyond this landing area, there is nothing but a trackless mountain massif which is difficult or even impossible of penetration by such a force. However, the use of airborne troops may offer an effective means by which passes over a natural barrier may be seized".

g. Wave and Surf Climate:

Waves are an aspect of weather and like weather show average values over long periods which may be designated as the "wave climate" of a certain area of the ocean.

Just as the weather climate of a region is the overall controlling factor so is the wave climate the controlling factor for a region. The wave climate has to do with such questions as "what are the normal wave, heights, periods and frequencies in the Persian Gulf during January". Overall planning must depend upon the most likely conditions; hence, upon wave climate.

The surf climate depends upon the wave climate and the hydrography. Because of this certain beaches may have a very mild "surf climate" even though the "wave climate" may be severe.

h. Hydrography:

"Hydrography is defined by Bowditch as 'that branch of science which

deals with the measurement and description of the physical features of that portion of the earth's surface which embraces the oceans, seas, lakes, rivers, and other waters and their adjoining coastal areas with special reference to their use for the purpose of navigation". From this it may be said that hydrography as pertains to amphibious operations embraces the detailed charting of the offshore areas in the objective areas to determine the suitability or unsuitability of the beaches for landing purposes and subsequent operations. In the past, hydrography has been applied solely for the purpose of safe navigation, but in connection with amphibious operations it has become a much more exacting study than ever before."

"Considerations of the hydrography of the landing area are of great importance, both in the planning phase and during the actual execution of the landing. The beach proper may be ideal for an amphibious operation, but it has no value unless the landing craft can land the troops, equipment, and supplies which are required for the execution of the operation ashore. Certain elements of hydrography at a landing area may exert a critical influence on the success of the operation unless consideration is given to the beach location, the approaches to the beach, the beach gradient, surf, tide, currents, natural obstacles, and the nature of the bottom. Reconnaissance of the target area must embody a study not only of the conditions on land but also of those conditions seaward of the shore line".

From USMC, Amphibious Manual, AM-2, 1951.

One of the main uses of hydrographic data is in applying refraction drawing techniques to predict surf conditions from deep water wave forecasts. (or hindcasts).

i. Beach Trafficability:

One of the most important factors in an amphibious landing is that of getting vehicles from the landing ships or craft across the beach and onto hard terrain. The present state of knowledge of beach trafficability and stabilization is set forth in detail in Section V of this Manual.

It has been found that from an operational standpoint the trafficability of a beach is primarily related to the steepness of the beach face and to the moisture content of the material. In regard to sand beaches the flatter the beach face the better is the trafficability, and also the greater the moisture content the better is the trafficabiltiy. Hence, from a trafficability standpoint a sand beach offers most favorable conditions during a falling tide (using the wet section) or after a heavy rainfall.

j. Tides:

(1) "Information Required for Amphibious Operations: In amphibious operations it is necessary to know the times of high and low tide and the range of tide. This is necessary because the stage of tide affects the width of the beach and, thereby, the type of surf, the depth of water over sand bars and reefs, the effectiveness of underwater obstacles, the stranding of landing craft, the width of exposed beach to traverse, and the requirements for special naval gear to facilitate debarkation".

From USMC, Amphibious Manual, AM-2, 1951

(2) Effect of Tide on Flat Underwater Gradient: "The tide level with reference to the beach gradient and its influence on landing craft is of vital importance. When a relatively large tide range occurs on a gently sloping beach, the flow of water or the speed with which it

rises or descends on the beach may be rapid enough to strand landing craft on a dry bottom before they are able to retract. Unless this possibility is calculated, a critical number of landing craft may be rendered inoperative until the next rise of tide".

From USMC, Amphibious Manual, AM-2, 1951.

(3) Irregularities of Sea Floor: "When, in addition to a flat surface, the beach also has considerable irregularities, a dropping of the tide may cause boats to ground far off the beach proper, thus compelling the personnel to debark and wade a considerable distance to shore through pools of water left on the beach. If these pools are deep a considerable loss of equipment must be anticipated".

From USMC, Amphibious Manual, AM-2, 1951.

(4) Preferable Time of Landing: "In general, when defensive obstacles do not exist, it is preferable to make the landing on a flood tide in order to facilitate beaching and retraction for the initial and subsequent waves of landing craft. This is particularly true if the tidal range is great and the gradient is very gentle. Maximum advantage may be taken of the rising water to overcome the possible hazard of gradient and beach bottom if the time of landing is established as 2 or 3 hours prior to high tide".

From USMC, Amphibious Manual, AM-2, 1951.

(5) Effect on Trafficability: In regard to trafficability, it is better to land during a falling tide.

k. Fleet Hazards:

(1) Exposure to Winds: "In open roadsteads, a slight shift in the wind may bring about a halt in the unloading. One of the primary naval considerations must be shelter from unfavorable weather conditions".

From USMC, Amphibious Manual, AM-2, 1951.

(2) "Maneuver Area Free of Navigational Hazards: There must be a maneuver area which is free of navigational hazards. Navigational hazards may include rocks, shoak, tide rips, and currents. It is important that it be clear both in the approaches to the landing area and in those areas which are adjacent thereto."

From USMC, Amphibious Manual, AM-2, 1951.

l. Holding Ground:

There is little information available-either intelligence methods or the importance given to obtaining data-concerning the adequacy of holding bottom at prospective landing sites. In the Alaskan campaigns, several destroyers were stranded, and at Iwo Jima there was a great deal of difficulty in anchoring ships and torpedo nets. These experiences show that information on the type and quality of holding bottom is vitally needed in planning any landing operation.

2. Classification of Surf:

a. Introduction:

Before amphibious commanders can make decisions regarding amphibious operations it is important that they be furnished with the necessary oceanographic data, systematized in a logical form. Part of the vital information is a description of the surf, either as observed or forecast, or both.

The Combined Surf Code as contained in Hydrographic Office Publication No. 206, 1946, "Radio Weather Aids to Navigation", and Commander Amphibious Force, Pacific Fleet, Circular Letter L-63 of 16 December 1950, set forth a code form to be used for describing surf conditions at a landing beach.

Examination and study of the above-listed publications by members of the Wave Research Group at the Universtiy of California has brought forth certain suggestions, opinions, and recommendations as to the data required to transmit an adequate coded description of a surf zone. The basis of recommended changes and suggested revision of the present code form is the information gained through wave research at this activity during the past few years.

The intent of this section is

(i) to set forth recommendations and proposals for changes to the present code,

(ii) to present a revised edition of the Combined Surf Code in accordance with the proposed changes and recommendations, and

(iii) to contribute new ideas and concepts which may be of value in future revisions of the Code Form.

b. Comments and Recommendations:

(i) An up-to-date Manual of Instructions for Observers should be published in conjunction with the Combined Surf code. The Manual of Instructions should contain such information as (1) minimum duration of observation periods, (2) latest information on the types and use of wave measuring devices, (3) general features involved in Surf Observation and new techniques in observation and measurement, and (4) a pictorial presentation of the different conditions for use as a guide in making wave observations.

(ii) The method of observation, i.e., aerial, ground, etc., is not indicated in the code message. It is believed that knowledge of the observation method employed would be of value to recipients of the message.

(iii) The duration of the observation period is not indicated. Recipients of the message need to know the length of the observation period so that they may weigh the value of the coded information. Obviously information received from an

observation period of one hour or more would tend to be more valid than a period of five or ten minutes.

(iv) Breaker height is more adequately described by the "significant" breaker height (i.e., the average of the highest one-third of the breakers) rather than by the average height of the breakers.

(v) The period should be the average period of the significant breakers and should be transmitted directly in seconds.

(vi) No provision is made to indicate more than one set of waves. Secondary wave trains often greatly influence the surf pattern, surf intensity, and littoral currents. This, in turn is an influencing factor on landing craft operating in the surf area. Secondary wave trains, if present, should be described in clear text.

(vii) A description of the breaker type should be given.

(viii) A more adequate description of the surf zone should be given.

(ix) The extremeties of the observed area and the lengths of the wave crests are not mentioned. This factor is considered important, particularly from the view point of selection of a site for a landing force.

(x) $L_b$ (previously $L_s$) should be breaker wave length instead of initial wave length.

(xi) In order to properly evaluate observations, or to make forecasts of breakers, it is necessary to have information of the hydrography and tides of the beaches. Distance of outer breakers from shore, character of surf zone, etc. depend upon the tidal range and stage. Hence, the tidal range of the beach in question should be coded into the message and the stage of the tide during the observation interval or the period for which the forecast is valid must be sent in clear text.

(xii) The tide range should be coded.

(xiii) The presence of rip currents through the surf ("surf rips") as well as a description of their magnitude and location should be sent in clear text as an addition to the coded message.

(xiv) Unusual surf phenomena which cannot be classified by code description should be sent in clear test as an addition to the coded message.

(xv) The effect of surf and swell conditions upon landing craft operations should be left out. A reliable estimate of this requires highly specialized personnel .

c. Conclusions:

(i) It is considered that the Combined Surf Code as contained in COMPHIBPAC CL, L-63 of 16 December 1950 does not include all features necessary to adequately transmit a coded description of surf conditions at a landing site.

(ii) That a pictorial presentation of the surf characteristic reported in the code form is a prime necessity for instruction and use of persons making and reporting surf observations. This is necessary to standardize the descriptions.

(iii) That the code form be expanded to contain additional supplemental data groups, in order to more fully describe the surf area.

(iv) That the code form be revised to allow inclusion of clear text messages to describe important conditions that are not covered by the coded message.

(v) That special consideration should be given to the code to determine if its use is satisfactory for intelligence and reconnaissance of enemy-held beaches.

(vi) That the Combined Surf Code should be supplemented by a coastline code classification to adequately describe the landing site.

(vii) That emphasis be placed on training observers in the use of the code form and proper methods and techniques of surf observation.

d. Proposed Code:

This code is used to give a description of surf condition at a landing beach. The code is so arranged that it can be used from the ground or from aircraft and for forecasting. Separate code forms have been devised for observing and for forecasting. Each code contains one or two groups to indicate place of observation, a Date time group (YYGGN or $YYG_bG_bN$), a Primary data group $H_bH_bM_bTT$, a Secondary data group ($t_bd_bA_bL_bC_b$), and a Tertiary group. ($D_bS_sT_rPE$).

(1) Code Forms:

It is considered that 4 code forms are adequate for observation and forecasting procedures as utilized in conjunction with amphibious operations. Code forms are as follows:

(a) For Observing:

23456 $I_nIIII$ YYGGN $H_bH_bM_bTT$ $t_bd_bA_bL_bC_b$ $D_bS_sT_rPE$

23456 $OL_aL_aL_aL_a$ $QL_oL_oL_oL_o$ YYGGN $H_bH_bM_bTT$ $t_bd_bA_bL_bC_b$ $D_bS_sT_rPE$

(b) For Forecasting:

23456 $I_nIIIII$ $YYG_bG_bN$ $H_bH_bM_bTT$ $t_bd_bA_bL_bC_b$ $D_bS_sT_rPE$

23456 $OL_aL_aL_aL_a$ $QL_oL_oL_oL_o$ $YYG_bG_bN$ $H_bH_bM_bTT$ $t_bd_bA_bL_bC_b$ $D_bS_bT_bP_bE_b$

23456 is the Key group assigned to the code for observing surf.

NOTE: If there are no waves observed or forecast, the primary data group $H_bH_bM_bTT$ is five zeros (00000) and the secondary and tertiary groups are omitted.

(2) Position Groups:

$I_n$ is the number designating a particular continental list:

1 - European list
2 - Asiatic list
3 - African list
4 - Pacific list
5 - North American list
6 - South American list
7 - East Indies - Philippine list

The first three III are the station index number taken from the appropriate list designated $I_n$. If confusion is likely to exist as to whether the station is primary of supplementary the appropriate Combined Meteorological Committee group (18881) should be added preceding $I_n$.

The fifth figure (I) in the group will be 0 unless locally assigned to designate temporarily a reporting station of known position near an already established international station.

$OL_aL_aL_aL_a$. The 0 indicates that the position will be given by latitude and longitude rather than by international index numbers. $L_aL_aL_aL_a$ gives the latitude in degree and minutes.

$QL_oL_oL_oL_o$. Q gives the octant of the globe according to New International Code No. 70, and $L_oL_oL_oL_o$ gives the longitude in degrees and minutes with the figure for hundreds omitted when the longitude is 100° or more.

Symbol Q - Octant of Globe

| Longitude | Code No. | Longitude | Code No. |
|---|---|---|---|
| North Latitude: | | South Latitude: | |
| 0° W to 90° W | 0 | 0° W to 90° W | 5 |
| 90° W to 180° W | 1 | 90° W to 180° W | 6 |
| 180° W to 90° E | 2 | 180° W to 90° E | 7 |
| 90° W to 0° E | 3 | 90° E to 0° E | 8 |

Date and Time Groups (G.C.T.) YY designates the date of the months. GG gives the time of the beginning of the observation period to the nearest quarter hour. This is done by adding to the hour observation 25 for the quarter hour. (Examples: 01152 coded as 26; 2345Z coded as 98; etc.) $G_bG_b$ gives the time of the beginning of a forecast period in a similar manner.

The fifth figure, N indicates the type of message and method of observation. If N is 0 through 4 the message reports an observation and the method of observation is stated. If N is 6 through 9 the message is a forecast and the degree of confidence in the forecast is stated. No confusion should arise in detecting the correct message form for observing or forecasting since the absence or presence of the figure 0 readily indicates the correct form of the two possible code forms.

Table for N

| Code No. | Observation Type of Message | Method of Observation |
|---|---|---|
| 0 | Observation | Aerial |
| 1 | " | Ground |
| 2 | " | Small boats seaward of surf |
| 3 | " | UDT team in surf |
| 4 | " | Any combination of above |

Table for N

| | Forecasting | Degree of Confidence |
|---|---|---|
| 6 | Forecast | No specification |
| 7 | " | With confidence |
| 8 | " | Uncertain |
| 9 | " | Very doubtful |

(3) Primary Group -- $H_bH_bM_bTT$:

$H_bH_b$ - Average height of highest one-third of the breakers (the significant breakers) in the dominant wave train. Recorded directly in feet. If it is impossible to forecast or observe height with any accuracy, report as 99. Example: If the significant breaker height were 7 feet, it would be coded as 07. If the significant breaker height were 12 feet it would be coded as 12.

$M_b$ - The difference between the height of the maximum wave and the significant wave during the observation, or forecast, interval. Recorded as follows:

| Code No. | Difference, in feet, between significant breaker height and maximum breaker height |
|---|---|
| 0 | No surf |
| 1 | 0-2 feet |
| 2 | 2-4 " |
| 3 | 4-6 " |
| 4 | 6-8 " |
| 5 | 8-10 " |
| 6 | 10-12 " |
| 7 | 12-14 " |
| 8 | 14-16 " |
| 9 | Impossible to estimate |

Example: If the significant breaker height were 7 feet and the maximum breaker observed during the interval were 14 feet, then $M_b$ would be reported as 4.

TT - The significant wave period. Record directly in seconds. If it is impossible to forecast or observe the period with any accuracy, report as 99. Example: a 7 second wave would be recorded as 07 while a 12 second wave would be recorded as 12.

NOTE: If no waves are observed or forecast, the primary code group is 00000.

(4) Secondary Group - $t_b d_b A_b L_b C_b$:

$t_b$ - Type of breaker
$d_b$ - Bottom depth at point where outermost waves breaks
$A_b$ - Angle formed by breakers in dominant wave train with beach (Referred to observer on shore facing sea.)
$L_b$ - Breaking wave length: the distance from the crest of the outermost breaker, form line or wave shoreward.
$C_b$ - Longshore current: magnitude and direction.

Table for $t_b$

| Code No. | Breaker Type |
|---|---|
| 0 | Calm |
| 1 | Surging |
| 2 | Borderline between surging and plunging - some of both. |
| 3 | Almost all breakers are plunging. |
| 4 | 70% of breakers are plunging, 30% are spilling. |
| 5 | Half of the breakers are plunging, half are spilling. |
| 6 | 70% of the breakers are spilling, 30% are plunging. |
| 7 | Nearly all of the breakers are spilling. |
| 8 | Breakers appear to be breaking in a manner half way between plunging and spilling with a tendency to plunge. |
| 9 | Breakers appear to be breaking in a manner half way between plunging and spilling with a tendency to spill. |

## Table for $d_b$

| Code No. | Bottom Depth at Point Where Waves in Dominant Wave Train Begin Breaking (feet) |
|---|---|
| 0 | Calm |
| 1 | up to 4 feet |
| 2 | 4 - 6 |
| 3 | 6 - 8 |
| 4 | 8 - 10 |
| 5 | 10 - 12 |
| 6 | 12 - 16 |
| 7 | 16 - 20 |
| 8 | More than 20 |
| 9 | Depth of breaking not measured, or impossible to estimate with any degree of accuracy. |

## Table for $A_b$

| Code No. | Angle which Breakers of Dominant Wave Train form with beach Measured with Reference to Observer on Beach facing the sea. |
|---|---|
| 0 | Approaching perpendicular to shore; no angle formed (If $H_bH_b$ is 00 this means no waves). |
| 1 | Approaching from the left and forming angle of 0-5° |
| 2 | Approaching from the left and forming angle of 5° - 10° |
| 3 | Approaching from the left and forming angle of 10° - 15° |
| 4 | Approaching from the left and forming angle greater than 15° |
| 5 | Approaching from the right and forming angle of 0-5° |
| 6 | Approaching from the right and forming angle of 5° - 10° |
| 7 | Approaching from the right and forming angle of 10° - 15° |
| 8 | Approaching from the right and forming angle greater than 15° |
| 9 | Impossible to estimate with any degree of accuracy. |

Table for $L_b$

| Code No. | Breaker Wave Length-Distance from crest of outermost breakers to crest of preceding breaker on foam line of wave shoreward. |
|---|---|
| 0 | Only a single line of breakers near edge of beach. |
| 1 | Up to 30 feet |
| 2 | 30 to 60 |
| 3 | 60 to 100 |
| 4 | 100 to 200 |
| 5 | 200 to 300 |
| 6 | 300 to 500 |
| 7 | 500 to 1000 |
| 8 | 1000 feet or over |
| 9 | Impossible to estimate with any degree of accuracy |

Table for $C_b$

| Code No. | Longshore (littoral) current, Velocity and Direction-Relative to an observer on the beach facing the sea. |
|---|---|
| 0 | No longshore current |
| 1 | Up to 1 knot (from left to right) |
| 2 | 1 to 2 knots (from left to right) |
| 3 | 2 to 3 knots (from left to right) |
| 4 | More than 3 knots (from left to right) |
| 5 | Up to 1 knot (from right to left) |
| 6 | 1 to 2 knots (from right to left) |
| 7 | 2 to 3 knots (from right to left) |
| 8 | More than 3 knots (from right to left) |
| 9 | Not measured, or impossible to estimate due to variability. |

(5) Tertiary Group - $D_b S_s T_r PE$:

$D_b$ - Distance of outermost breakers (not white caps) from shore.
$S_s$ - Description of surf zone; i.e., wave breaking on shore, over one bar etc.
$T_r$ - Tide range for interval during which observations were made or for which forecasts are made.
P - Duration of observation or forecast interval, measured from the beginning of the interval as given by the date-time group. Also tendency of surf to increase, decrease or remain constant during observation period.
E - Length of coast covered by observation or forecast, with the point of observation (or forecast) at the center. Also, the crest length of the waves (distance along the crest).

## Table for $D_b$

| Code No. | Distance at Outermost Breakers from Beach-Measured from the Beach at the Point of Backrush. |
|---|---|
| 0 | No breakers up to backrush (at edge of beach) where each wave breaks once or not at all. (If $H_bH_b$ is 00 this means no breakers) |
| 1 | 0 - 100 feet |
| 2 | 100 - 200 feet |
| 3 | 200 - 400 |
| 4 | 400 - 800 |
| 5 | 800 - 1200 |
| 6 | 1200 - 1600 |
| 7 | 1600 - 2000 |
| 8 | Over 2000 feet |
| 9 | Impossible to estimate with any degree of accuracy. |

## Table for $S_s$

| Code No. | Description of Surf Area |
|---|---|
| 0 | Waves Break on Shoreface |
| 1 | Single line of breakers (less than 10 yds. wide) followed by relatively unbroken water inshore. |
| 2 | Single line of breakers with foam lines in direction of beach. |
| 3 | Single outer line of breakers with lower breakers inshore. |
| 4 | Outer line of breakers, then an area of swell then another line of breakers with foam lines and smaller breakers inside. |
| 5 | More or less regular lines of breakers with waves reforming and breaking again in shore. |
| 6 | Outer line of breakers; does not extend along entire surf area; more or less continuous inner lines of breakers. |
| 7 | Breakers in oblique segment due to secondary system of waves coming from an angle to the dominant swell; secondary train usually is short period. |
| 8 | Confused breakers covering entire surf area. |
| 9 | Not described by any of the above types. |

Table for $T_r$

| Code No. | Tide Range |
|---|---|
| 0 | Less than 2 feet |
| 1 | From 2 to 4 feet |
| 2 | " 4 to 6 " |
| 3 | " 6 to 8 " |
| 4 | " 8 to 10 " |
| 5 | " 10 to 14 " |
| 6 | " 14 to 18 " |
| 7 | " 18 to 22 " |
| 8 | Greater than 22 feet |
| 9 | Unknown |

Table for P

| Code No. | Observation Duration of observation period | | Forecast Duration for which forecast is designed commencing at from given DTG |
|---|---|---|---|
| 0 | Less than 5 minutes | | 0 - 6 hours |
| 1 | 5 - 10 minutes | | 6 - 12 hours |
| 2 | 10 - 12 minutes | | 12 - 18 hours |
| 3 | 20 - 30 minutes | | 18 - 24 hours |
| 4 | 30 minutes to 1 hour | surf intensity | 24 - 36 hours |
| 5 | More than 1 hour | decreasing | 36 - 48 hours |
| 6 | 30 minutes to 1 hour | surf intensity | 48 - 60 hours |
| 7 | More than 1 hour | increasing | 60 - 72 hours |
| 8 | 30 minutes to 1 hour | No evident tend- | 72 - 84 hours |
| 9 | More than 1 hour | ency to change | 84 - 96 hours |

Table for E

| Code No. | Length of Coast Covered by Observation with Point of Observation at center | Length of Wave Crests |
|---|---|---|
| 0 | Less than 1/2 mile | Waves short-crested |
| 1 | 1/2 - 1 mile | |
| 2 | More than 1 mile | |
| 3 | Less than 1/2 mile | Wave Crest medium in length |
| 4 | 1/2 - 1 mile | |
| 5 | Over 1 mile | |
| 6 | Less than 1/2 mile | Wave Crest long |
| 7 | 1/2 mile - 1 mile | |
| 8 | Over 1 mile | |
| 9 | Distance not known | |

3. Classification of Shorelines:

Before amphibious commanders can make decisions regarding amphibious operations, it is important that they be furnished with the necessary oceanographic data, systematized in a logical form. Part of the vital information is a description of the shoreline. At the present time there

is no adequate classification of shorelines, from the standpoint of amphibious operations. The development of such a classification is one of the most important remaining problems in this field.

Information should be included such as the protection afforded by headlands, offshore islands and reefs; exposure to prevailing swell; number of bars; length of beach, width of beach, composition of beach, and shape of beach; accesability; obstructions; etc.

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MANUAL OF AMPHIBIOUS OCEANOGRAPHY

SECTION VIII: APPLICATION TO AMPHIBIOUS OPERATION

# C. SHIP TO SHORE PHASES

BY
LT. D. A. PATRICK, CEC, USN AND LTJG D. A. ELLIS, CEC, USN

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# C. SHIP TO SHORE PHASES

## 1. Some Special Aspects of Amphibious Operations:

### a. Introduction:

Amphibious Oceanography, compared to other engineering professions, is a relatively new development. The newness of the profession and also the wide variety of beach, sea, and weather conditions throughout the world make it impossible as yet to formulate reliable rules to cover all aspects of landing conditions and landing operations. Therefore the information presented in various sections of this Manual tends to be a general nature. It is based on data and knowledge gained from observation of selected portions of the world's coastlines. In considering the application of this information to a large scale amphibious operation in remote areas of the world where beach, sea, and weather conditions are vastly different, it is obvious that additional data, research, and development are needed.

The intent of this section is to discuss various problems and conditions which may arise or are now inherent in amphibious operations, but which apparently have received little attention in the past. The problems may appear slight compared to other requirements of selecting landing sites, but additional data and knowledge of the factors would be beneficial for planning purposes.

### b. Endurance of Personnel:

(1) Seasickness: An unknown percentage of personnel are subject to seasickness when exposed to various sea conditions for varying lengths of time. It would be desirable to know the percentage of troops in a landing craft who would become seasick under various wave conditions for varying lengths of travel time of the craft from ship-to-shore. Of even greater importance would be to know to what degree seasickness reduces the combat capability of the trooper when he reaches the beach.

(2) Effect of Combat Load: Assuming a landing craft grounds before reaching the beach face, and that the trooper must wade ashore loaded for combat, it would be desirable to know how far, at what water depth, and through what type of surf, a combat-loaded trooper can go without seriously impairing his fighting ability on reaching shore. If casualties due to surf conditions must be expected, a correlation between percent casualties under various surf conditions, distance, topography, and water depth would be valuable. Apparently the effects of so-called "oceanographic" casualties cannot be considered separately from the combat casualties because these two types, though not serious in themselves, may combine to be disastrous. Col. S. L. A. Marshall (Reference VIII C-6) after detailed study of the Omaha Beach landings at Normandy on 6 June 1944 concluded that "weight and water - directly or indirectly - were the cause of the greater part of our losses at the beach." Although the surf conditions were not particularly hazardous at the time of the assault, many soldiers who either stumbled or were wounded slightly could not rise - apparently because of heavy packs - and were overcome by the tide which was advancing as fast as a slowly walking man.

(3) Cold Climate: There have been numerous publications relating to survival in cold climates of the polar regions. These indicate that personnel can survive the frigid waters for only a matter of minutes unless rescued promptly.

An Amphibious Operation in cold climate, although difficult, is not out of the question. In such operations, it appears that the severest effects on personnel of extreme cold are probably during the ship-to-shore phase of

unloading troops and supplies. Landing boats would surely afford a lesser degree of weather protection than could be found on the beach.

Data should be obtained on landing craft operation in icy waters, and training manuals prepared for the use of troops. Such questions as these should be answered:

(i) Protection of troops from sea spray,

(ii) Physical endurance of personnel to extreme temperatures,

(iii) Reduction of combat capability due to exposure in landing craft while enroute to the shore,

(iv) Instructions for survival in icy waters,

(v) Reduction in wave height due to damping action of ice floes, etc.

c. Landing Ships, Craft and Vehicles in Cold Water Operations:

It is understood that studies and tests are being conducted to develop equipment and materials suitable for operation in cold climates. Special cold weather lubricants have been developed, snow vehicles are in use, and survival instructions for personnel ashore have been formulated. However, little information seems to be in circulation about the operational characteristics of landing ships and craft encountering and maneuvering in the vicinity of icy shores, pack ice, fast ice, etc.

The Arctic Manual, published by the U. S. Army Air Corps in 1940, (Reference VIII C-9) states that:

"For sea navigation, scattered ice will have more advantages than disadvantages. However rough the ocean where it is open, as soon as you get in among the ice you have no more trouble with the waves. The floes near shore are usually scattered. Few of them are bigger than a city block in area and there likely are between them half-mile patches where you can sail through water that is smooth even with a stiff breeze."

(Reference is also made to:)

"...a small boat, 50 feet in length, drawing 4 feet 2 inches loaded having a center board in lieu of a keel which could be withdrawn into the body of the ship. When her commander saw ice closing in on the ship where there seemed to be no chance of getting out of the way entirely, he would select in the neighborhood some big ice cake that sloped to the water's edge on one side. He would then steam full speed ahead against this floe. The bow was so shaped that, instead of hitting the ice a hard blow she would slide up on it, standing level because of a flat bottom. Thus by her own power, she was able to put herself halfway on top of the ice. The crew were prepared to jump out, fasten an anchor into the ice, and with blocks and tackle to haul the ship entirely up on the floe and escape the closing in of other ice floes."

It appears that important data could be compiled from case histories of the numerous expeditions into polar regions and from reports such as the Artic Manual. Such information would be helpful in developing operational procedures for landing ships and craft in icy regions. Information concerning the protection afforded from damping of ocean waves by ice floes would be

an asset to landing craft operators as well as to the general field of amphibious oceanography.

d. Calming Waves By Use of Oil:

Investigations, reports of mariners, tests, and experiments produce a variance of opinions concerning the merits of using oil to dampen and reduce ocean waves. All investigations seem to agree that in a "sea" the waves are noticeably reduced by use of oil slicks. Another conclusion is that oil films tend to prevent or hinder inception of waves and to prolong the breaking effect. However the above conclusion is limited to small wind waves (chop).

For large ocean waves it is difficult to see how an oil film can have any noticeable effect, since the forces involved are so much greater than the surface tension forces which are responsible for the damping effect of the oil. (Gibson, 1945. Reference VIII C-2) However mariners' reports sometimes laud the use of oil in launching life boats in heavy seas. Additional research is needed to solve this problem.

The wave-damping effect of oil and its effective use in an amphibious operation appears to have little value. However, special types of operations (such as a night raider party launched in rubber boats from submarines) might possibly benefit by using oil to produce calmer seas.

A summary of the calming effect of waves by oil is well stated in the Hydrographic Office Publication: Wind Waves at Sea - Breakers and Surf (1947, Reference VIII C-13) and the following paragraph is quoted therefrom.

> "The calming effect on waves of a film of oil has been known for ages; Pliny, for example, mentioned it in the year 77 A.D. And every work on seamanship includes directions for its use in storms. Briefly, oil extinguishes the smaller wavelets, and---its especial value---prevents the waves from breaking. It is not clear just how this is brought about, for the older view that its effects are due to lessened friction with air seems not to be correct, while it is doubtful whether more recent explanations, based on the relative viscosity and surface tension of oil as compared with water, are adequate. But in any case, a film of oil spreading over storm seas, so lessens the grip of the wind on them, that it immediately causes them to assume the character of swells, and thus renders them much less dangerous. Oil, however, does not interfere appreciably with the development of surf, for in this case it is not the immediate effect of the wind that is responsible for the formation of breakers. But oil may be of real assistance if it is necessary to attempt landing during onshore winds strong enough to cause a breaking sea off shore, for its use may prevent the waves breaking until they reach the surf line proper."

e. Visibility:

Weather visibility conditions and resultant effects on a landing operation must be carefully investigated. Night landings, or landings executed under adverse visibility conditions are difficult unless properly planned for. It seems that a balance between good and poor visibility conditions is desirable. Low visibility may be utilized for screening and protecting the operation from the enemy. On the other hand, visibility must not be poor enough to hamper aerial support, Naval bombardment, and the progress of landing craft and ships to and from the beach.

Narratives of landings in the Alaskan area during World War II tell of the difficulties encountered during landings when the weather visibility was poor (dense fog). Collisions between ships occurred; landing-craft operators had trouble landing at the right beach area and then returning to their support ship; and naval bombardment and aerial support was hampered. As a result, destroyer radar was used in directing the entire landing operation. Its use proved very satisfactory.

The best preparation for landings under adverse visibility conditions appears to be proper and adequate training exercises which simulate conditions of the actual landing operation. Perhaps radar could be used in smaller craft such as picket boats to direct the ship-to-shore movement.

f. Holding Ground:

There is little information available - either on intelligence methods or the importance given to obtaining data - concerning the adequacy of holding bottom at prospective landing sites. In the Alaskan campaigns, several destroyers were stranded, and at Iwo Jima there was a great deal of difficulty in anchoring ships and torpedo nets. These experiences show that information on the type and quality of holding bottom is vitally needed in planning any landing operation.

2. Debarkation on an Open Coast:

a. Introduction:

In this section, "Debarkation on an Open Coast" includes two operations:

(i) The off-loading of landing craft, equipment personnel and cargo from support ships offshore, and

(ii) The progress of these landing craft and vehicles to the surf zone.

In the debarkation phase of an amphibious operation, many different type boats, lighters and craft are used, and these are subjected to more or less unprotected waters for long periods. The success, therefore, of any landing operation will depend largely on the sea and weather conditions prevailing during the off-loading operation. While it is possible to predict sea and weather conditions for a planned debarkation several days in advance, the difficulty is that it is not always possible to carry out debarkations on schedule. Often, emergencies will necessitate debarking at a moment's notice, when the weather and sea are very rough.

It is therefore most important to determine the capacities and limits of the various craft and vehicles used in the debarkation. What is safe, and what dangerous for each type? Which type can stand the most punishment? To answer these questions, much more data are needed on such factors as: limitations of wave height, wind and current velocities, and general sea conditions. For example, small boats may operate successfully in low waves, but function very poorly in waves over a certain height, and not at all in still higher waves.

Especially for debarkation in rough weather, it would be most helpful to be able to predict percent casualty of craft that might be expected for each type of sea condition. Data are needed on performance of craft and vehicles - both during operations alongside ship, and in receiving and discharging cargo and personnel.

To date, no definite answers to these problems can be given. Experience and data are lacking on sea and weather conditions; and on the performance of the equipment used in debarking operations.

b. Scope of This Section:

This section summarizes the available data in the hope that it may prove useful, particularly in training programs. It is limited to:

(i) A pictorial presentation of various procedures used in the debarkation phase, and

(ii) A recommendation for further observation and collection of data - about general safety limits for craft when receiving and discharging cargo alongside ship; and also when operating in open waters.

A start on the problem has been made by a firm of consulting oceanographers working with oil companies on the Gulf Coast of the United States. The accompanying Table is based on their studies, and shows the performance limits (in terms of wave heights) of several type boats for a number of off-shore drilling operations.

The pictorial presentation of debarkation preceedings is self-explantory. All phases of the operation are not included, as no photographic illustrations of them were available.

In future studies, special attention should be given to observing and collecting data on the pitching and heaving behavior of craft tied alongside ship, and used in debarking troops and supplies.

c. Suggested Program for Studying Debarkation Factors:

A program to evaluate effects of sea conditions on the craft and to improve methods of unloading, should make a study of the following:

(i) Type of craft alongside ship

(ii) Type of support ship

(iii) Wave description:

- (a) Height
- (b) Period
- (c) Direction

(iv) Currents:

- (a) Velocity
- (b) Direction

(v) Mooring of craft alongside ship:

- (a) General description of method used
- (b) Type and number of fenders used

(vi) Pitching and heaving of craft:

(a) Maximum degree of pitching and heaving
(b) Height of rise and fall of craft on waves

(vii) Time duration craft is alongside ship for cargo loading:

(a) Type of cargo being unloaded
(b) Method of unloading
(c) Estimated tonnage loaded into craft

(viii) Wind:

(a) Velocity
(b) Direction
(c) State of wind: gusty, steady, or unsteady

(ix) Damage inflicted to craft under observed sea conditions.

(x) Damage occurring to craft while clearing or coming alongside ship under various sea conditions.

(xi) Effect of making a lee in unloading procedures:

(a) Reduction of wave height
(b) Reduction in wind velocity
(c) Reduction in heaving and pitching of craft on the lee side.

d. Conclusion:

If the above information were available, it would then be possible to formulate safe limitations of sea and weather conditions, and to prepare a guide for using various type craft and vehicles in Debarking Operations.

e. Appendix:

Table: Generalized Performance Data for Marine Operation*

| Type of Operation | Wave Heights (in feet) for: Safe, Efficient Operation | Marginal Operation | Dangerous and/or Inefficient Operation |
|---|---|---|---|
| Deep Sea Tug: | | | |
| Handling oil and water barge | 0-2 | 2-4 | >4 |
| Towing oil and water barge | 0-4 | 4-6 | >6 |
| Handling derrick barge | 0-2 | 2-3 | >3 |
| Handling and towing LST-type vessel | 0-3 | 3-5 | >5 |
| Crew Boats 60-90 feet in length | | | |
| Underway | 0-8 | 8-15 | >15 |
| Loading or unloading crews at platform | 0-3 | 3-5 | >5 |
| Supervisor's Boats--Fast Craft 30-50 feet in length | | | |
| Underway at cruising speed | 0-2 | 2-4 | >4 |
| Loading or unloading personnel at platform or floating equipment | 0-2 | 2-4 | >4 |
| LCT-type Vessel and Cargo Luggers | | | |
| Underway | 0-4 | 4-5 | >5 |
| Loading or unloading at platform | 0-3 | 3-4 | >4 |
| Loading or unloading at floating equipment | 0-4 | 4-5 | >5 |
| Buoy Laying (using small Derrick Barge) | 0-2 | 2-3 | >3 |
| Platform Building | | | |
| Using ship-mounted derrick | 0-4 | 4-6 | >6 |
| Using large derrick barge | 0-3 | 3-5 | >5 |
| Pipe-line Construction | 0-3 | 3-4 | >4 |
| Gravity-meter exploration using surface vessel (limiting conditions caused by noisy instrument becoming noisy) | 0-4 | 4-6 | >6 |
| Seismograph Exploration using craft under 100 feet in length | 0-6 | 6-8 | >8 |
| Large Amphibious Aircraft (PBY) | | | |
| Sea Landings and Take-offs | 0-1.5 | 1.5-3 | >3 |
| Boat-to-plane transfer operations in water | 0-1 | 1-2 | >2 |
| Small Amphibious Aircraft | 0-1 | 1-2 | >2 |

* According to Glenn, A. H., (1950, "Progress Report on Solution of Wave, Tide, Current and Hurricane Problems in Coastal Operations", Oil and Gas Journal, June, 1950 (Reference VIII C-3).

3. Assault Waves:

When the troops have been debarked and have proceeded to the line of departure, the question of whether or not the landing will proceed of course virtually has been decided irrevocably. The problem, then, is to make the best of the situation. Knowledge and skill on the part of the control officers and the individual coxswains will be more valuable for the marginal operations - those operations where the units must undergo certain risks.

With regard to the assault waves, tactical considerations require that a large number of small units be employed. But, in general, insofar as craft casualties from oceanographic conditions are concerned, the larger the craft the better. For the assault phase itself, the main consideration is the number of craft which will reach the beach and successfully discharge the load. In this connection, three oceanographic considerations are important:

(i) Oceanographic casualty statistics

(ii) Wave variability

(iii) Changes in tide stage

a. Oceanographic Casualty Statistics:

The casualty statistics of importance to the assault phase alone are those which will prevent the craft from carrying out its mission to the beach itself, but do not include the return to the debarkation area. In general, landing casualties will be more critical among vehicles such as the LVT than among the boats because the LVT are more apt to pitch over and be lost in the surf before beaching. Under certain surf and beach conditions the number of LCVPs and LCMs which broach may be so large as to preclude later boat waves.

b. Wave Variability:

If a wave recorder is in operation before and during the assault, this information combined with knowledge on wave statistics may enable the control officers at the line of departure to give the assault waves some advantage by proper timing of the beaching run on the basis of wave variability. Many factors will determine whether or not such delicate controls should be attempted - such as location of the pressure pickup of the wave recorder and whether or not any surf pattern can be observed on the record. Sufficient information is not available at present to say whether the high waves which pass over a recorder offshore maintain their identities as they travel shoreward to form the breakers. With present means of control of individual craft and present equipment, wave variability probably cannot be used to advantage during the assault waves. The additional instructions to individual craft which would be necessary would probably create more confusion than they would eliminate.

c. Changes in Tide Stage:

In general the landing sequence is facilitated if the assault waves beach about 2 or 3 hours before high tide (AM-2, 1951)*. The importance of rapidly unloading boats cannot be over emphasized. In this manner the time the craft is beached can be reduced to the minimum and the change in tide stage should have little effect. (The possibility of broaching is also reduced.) However, if some craft have trouble at the beach and cannot retract immediately, the rising tide should be an aid in that the craft will not be left high and

* Reference VIII C-12

dry and will be available for subsequent waves. If the tide stage has been predicted accurately, it should cause little unforseen trouble during the assault waves. However, if the prediction is inaccurate or doubtful as was the case at the Tarawa landings on 20 Nov. 1943, major difficulties may develop. At Tarawa the boats could not pass over the reef because the tide did not rise as high as predicted. Consequently, some of the troops had to transfer to LVTs and some had to wade ashore over fire swept water. Although the troops had been drilled for just such an emergency, the results still were grave. Possible causes for errors in tide prediction have been discussed in the section of this manual on Tides and about all that can be said is that if the tide is known to be a critical factor adequate drill in alternative plans is mandatory.

4. Supply Phases:

That the supply phase must follow the assault phase on schedule is obvious. However, certain basic differences in these two phases can be observed. First, the scheduling of the supply waves can be a little more flexible because they are not tied in with the lifting of Naval gunfire and the cessation of air strafing along the beach. Secondly, in general these craft will spend more time at the beach because supplies cannot be unloaded as fast as troops. The time the craft are beached for unloading must be kept to a minimum to reduce numbers of broaching casualties and tide stranded craft. As with the assault waves, true amphibians are more advantageous than amphibious craft in that a critical line of demarcation need not be established at the water's edge. In past operations, however, sufficient numbers of amphibians have not been available to permit initial phases to be accomplished without the use of boats. If amphibious units must be used the larger craft and ships are more desirable. Unfortunately, the boats often are required before the beach-head is secured so that dispersal still dictates the use of large numbers of smaller units. Thus amphibian vehicles, and amphibious craft and ships will all have somewhat different characteristics and will be affected in different manner by the oceanographic factors mentioned for the assault phase, namely, oceanographic casualty statistics, wave variabiltiy and changes in tide stage.

a. Oceanographic Casualty Statistics:

Excluding enemy action and "mechanical failures occurring outside the surf zone and not caused by previous landings, all other casualties probably have surf as the ultimate cause. An experienced coxswain could land and retract without difficulty in fog or moderately high wind and mechanical failures would occur much less frequently were it not for the existence of the surf. The number of variables is so great and their nature so intangible that it seems desirable to ultimately classify the casualties under the three principal headings Beach, Craft and Surf and allow the scatter of the data to represent the effect of the other variables...

"In general, the height of the breaker appears to be the predominant characteristic of the surf in relation to casualties but other features are important. Variability in height, period and the strength of the littoral current exert a strong influence and may predominate at times. High but variable surf which exhibits periods of relative calm may not be as difficult for an experienced coxswain as lower but more uniform surf of shorter period. Low waves of short period striking the beach at an angle may generate a littoral current which will cause more casualties than higher waves which are

parallel. Conditions in the surf are so complex and change so rapidly that it is impossible to obtain a complete and accurate record of the circumstances surrounding any casualty."*

The landing vehicles in present use, the LVT and the DUKW, are most apt to become oceanographic casualties during the passage through the surf. Therefore, from an oceanographic standpoint, the probable number of vehicles which will reach the beach is the same for the assault and supply phases and the probable number available for reuse will be about the same as the number expected to beach. For landing craft and lighters, however, the probable number which will beach successfully is often much different than the probable number that will retract successfully, i.e. the number available for subsequent waves. For these craft the majority of casualties occur on the beach itself by broaching. Therefore, the number which will successfully discharge their loads on the beach probably will be much larger than the number which will retract and be available for subsequent waves. An important secondary effect of craft broaching is that they clutter up the beach and reduce the flow of traffic. Some graphs of broaching casualties of LCVP and LCM are reproduced in Figure VIII C-16. Similar graphs are not available for larger craft and ships, but broaching casualties will generally be less than for the smaller craft. The broaching casualties can be reduced if shore crews and equipment attempt salvage promptly and concentrate on freeing the easiest cases first. Stern lines are much more effective than bow lines in preventing broaching casualties as shown in Figures VIII C17 and 18.

b. Wave Variability:

A critical zone for units passing from ship to shore is the surf zone and more particularly the breaker zone. As noted in the discussion of assault waves, means are not available at present to predict accurately the arrival time of lulls in the surf, even with the use of wave recorders. Although subsequent research may develop means of accurately predicting the arrival of the smaller breakers in the wave groups the practical use of this knowledge will depend on whether or not the arrival of the craft can be scheduled to coincide with the arrival of the lulls. The problem of controlling landings on the basis of wave variabilities becomes so complicated that entire waves of craft probably will not lend themselves to such control. However, certain liberties in scheduling often are permissible in supply phases that could not be allowed in assault phases. Thus if presence of surf variabilities can be announced to individual craft operators they may be able to exercise some prudence in traversing the surf zone, yet not delay the landing more than a minute or two. Surf variability is probably less important for the larger ships, but should be kept in mind when emergencies require that larger ships be beached in high surf.

c. Changes in Tide Stage:

Besides the indirect effects on character of the surf, etc., these changes need be considered more carefully for the supply phases than the assault phases. First the supply phases continue over much longer periods, and second, individual craft remain on the beach longer. Two other factors are so obvious as to hardly warrant mention: (i) That craft and ships unloading on flood tide rather than ebb tide are more apt to be able to retract, but that cargo must be immediately moved up on the beach, and (ii) the flatter the beach the faster will be the ebb or flood currents for a given tide range.

* Quoted from Reference VIII C-7.

5. Beach:

For this present discussion one need not draw a fine line of where ship to shore phases end and land phases begin. Consequently, trafficability, beach congestion, and water supplies on the beach itself have been considered.

a. Trafficability:

The details of trafficability have been discussed in Part V of this manual, (Beach Trafficability). Thus knowing some of the relationships between the various traffic phenomena and with the proper intelligence reports on hand one should be able to plan unloading schedules and handling of supplies on the beach. In at least two cases, Iwo Jima in 1945 and training Operation Miki in 1950, the trafficability was overestimated. As a result those craft and ships which discharged wheeled vehicles first, succeeded only in blocking the ramps. Further discharge of cargo could not proceed until a tracked vehicle from the beach pulled the stuck wheeled vehicle clear or the vessel retracted and beached again at another spot.

Steeper beaches will cause much trouble in respect to trafficability because vehicles must not only climb the grade, but steep ocean beaches characteristically are composed of relatively large rounded grains, which act as many individual small ball bearings. Mud beaches may give unanticipated trouble particularly if a crust is on them which may be broken through. Flat beaches may be perfectly stable for vehicles underway, but cannot support vehicles stopped for any length of time. Thus in spite of indications that wheeled traffic can be sustained on a given beach, a tracked vehicle should be the first off a landing ship or should be standing by to aid the wheeled vehicles the moment they start to debark.

b. Beach Congestion:

One of the critical links in the chain of logistic support for amphibious operations occurs at the transition from water to land. As a result most amphibious operations are characterized by a large mass of supplies on the beach itself with attendant confusion and congestion. The evils of beach congestion are obvious. Critical items may be on hand, but if they cannot be found or are inaccessible amidst the morass of material, they are worse than useless. Since supplies arrive in practically an unremitting stream a constant effort must be maintained to reduce congestion.

Many methods for reducing beach congestion are not affected too much by the oceanographic conditions, such as combat loading, indoctrination of shore party personnel, mechanized cargo handling using pallets, well maintained routes of egress, etc. But some of the methods for reducing beach congestion are affected by the oceanographic conditions at the site, such as:

(i) Use of amphibians to transport cargo directly to inland dumps.

(ii) Prompt salvage of broached and disabled craft on the beach.

(iii) Refueling LVTs and DUKWs at sea from ships or floating fuel stations.

(iv) Use of floating priority loads available at the beach on demand.

(v) Landing preloaded vehicles from landing craft and ships.

(vi) Proper and rational loading of craft and lighters in the debarkation area.

The oceanographic conditions (both present and predicted) at a site should be evaluated carefully so that some of the above methods for reducing beach congestion (and hence improving logistics) do not actually increase congestion. Particular consideration should be given to the use of the smaller craft and floating stockpiles as they may become a liability instead of an asset for certain conditions. The two extremes of beach congestion are illustrated by the Tinian operation where virtually no beach congestion existed and at Iwo Jima where the congestion was very critical.

At the Tinian operation, July 1944, the combined width of the two landing beaches was only 600 yards and the area which was trafficable for amphibian tractors and trucks was less than 200 yards. Yet two Marine Divisions landed over these beaches. Isely and Crowl (1951) (Reference VIII C-5) state (p. 352) that

> "...as a study in pure technical skill and amphibious virtuosity, the assault on Tinian excels any other landing in the history of the war."

And further, (p. 368)

> "...the successful execution of the supply plan against great natural odds remains in the end the most remarkable feature of Tinian operation. In the words of General Schmidt's supply officer: 'This operation was in many ways a remarkable demonstration of the fact that preconceived notions and amphibious doctrine can be altered radically on the spot. In effect, a reinforced corps was landed over less than 200 yards of beach and over a difficult reef, and was supplied throughout nine days of heavy combat without handling so much as one pound of supplies in the usual shore party manner. Everything rolled in on wheels. When a violent sea made impossible the landing of trucks, the DUKWs took over all supply, supplemented to a minor degree by incoming air evacuation planes bringing in rations. The troops never lacked what they required at the time it was required.'"

At Iwo Jima, Feb. 1945, in contrast to Tinian, beach congestion was critical. Much of the confusion on the beach was caused by enemy action, improper indoctrination of personnel, etc. But much of it was caused by the poor oceanographic conditions. Of the six methods listed above for reducing beach congestion, most were ineffective. But worse than inefficiency, the small craft and some of the lighters cluttered up the beach restricting the employment of the amphibians and larger landing vessels. As a result, on D plus 1 UDTs were landed to help clear the beach of broached and disabled craft.

Beach congestion probably cannot be eliminated; but in many cases it can be reduced. Amphibious practices and decisions which do not consider beach congestion along with other factors may prove to be very shortsighted.

c. Ground Water:

(1) Introduction: Portable water is essential to all military operations. It is especially important to amphibious operations where logistic support phases are critical (particularly at the landing site) because materials must usually be handled several times, often manually or with primitive equipment, and often under adverse conditions of weather, sea and the enemy. Marines manually unloaded water in drums in the Peleliu Invasion Sept. 1944, as shown in Figure VIII C-19. Cooling water for machinery is an important

secondary requirement for modern military forces and the upper allowable salinity limits are about the same as for potable water. Thus, any local source of potable water which can be quickly and safely exploited can contribute materially to the success of an amphibious mission by reducing the amount of supplies to be unloaded on the beach. Brackish water which is not necessarily potable is still a better source of raw water for distillation units than sea water.

Two main sources of water may exist close to a beach: water in ponds and lagoons, or ground water. In the case of sand beaches in humid climate water may enter the ground locally through precipitation. In dryer climates it may flow to the beach from the hinterland. Or the ground water may be a combination of both local precipitation and vadose flow.

Behind most beaches the salinity of the ground water drops off sharply. Thus if a satisfactory yield can be obtained, distillation units or filtration plants may get raw water only a short distance from the beach. The ground water situation on the beach can best be predicted by an engineer on the basis of rainfall and climatological data, geological maps and information, agricultural information of the hinterland (has the ground water been all used up by heavy pumping for irrigation), aerial photographs and other reconnaisance.

The importance of potable water to an amphibious operation (or any other modern military operation) cannot be overemphasized. Modern large scale landings are usually organized to include sizable amounts of fresh water in barrels, distillation units, filtration units and all the personnel necessary to operate and handle this equipment. The supply of these items is probably never great enough except for the minimum necessary used of potable water especially in the early stages of a landing. Someone must see, then, that the most efficient use of these facilities is made. One fundamental should be kept in mind: that raw sea water should normally be processed for potable water only as a last resort. Any possible source of fresh or brackish water should be evaluated and the best source for the raw water chosen.

The personnel assigned to the potable water units are of course presumed to know the necessary tests to perform and the most efficient methods of operation of their equipment. This section of the manual, then, will be confined to suggesting possible and probable sources of raw water in the immediate beach area which may or may not require processing to make it potable.

(2) Criteria for Potable Water: Three general criteria determine whether or not water is potable: suspended solids, dissolved solids, and organic content. The suspended solids will be visible if present in harmful amounts. On sand beaches in a shallow open well these suspended solids usually will settle sufficiently in a few minutes to permit use. For extensive pumping a fine sieve will probably be necessary on the intake line. The dissolved solids in beach ground water are often presumed to be those normally existing in sea water and in the percentages usually found in sea water. Thus the usual criterion is to express salinity as the weight of chlorine present as chloride per weight of water in parts per million (ppm) assuming that other salts (bromides, sulphates, etc.) are in corresponding proportions. Many situations can be visualized in which this assumption would not hold, but for the case of sand beaches, especially in humid areas, the assumption is reasonable. The average concentration of dissolved solids in sea water is about 35,000 ppm and the average concentration of chlorides is 18,000 to 20,000 ppm. The presence of 25 to 100 ppm chloride concentration in beach ground water generally indicates the presence of sea water.[1] The upper desirable limit for potable water is about 300 ppm of dissolved chlorides. Water of this salinity also may be used for short periods in vehicles.[1] This limit corresponds to a specific gravity of about 1.000375 which hardly can be measured readily in the field without special equipment. The upper limit of

chloride concentration recommended by the U. S. Public Health Service for potable water is 250 ppm. The organic content and quality of water should be tested using the standard Army filtration unit tests.[2] In emergencies, especially for engine cooling, acceptable water behind a beach perhaps could be tested by tasting. Even though acceptable potable water criteria may not be fulfilled, water of any salinity less than sea water would allow a corresponding reduction in amount of, fuel for and maintenance of portable distillation equipment, because the residue will be less and the cleanout periods will be longer apart.

(3) Published Information on Beach Ground Water: Comparatively little information has been published on beach ground water. Some information on beach ground water was obtained on the University of California Amphibious Oceanography project (Reference VIII C-8) and has been summarized graphically in Figures VIII C-20, 21, 22 and 23. These data were obtained from four specific locations as follows:

(i) beach backed by tidal lagoon connected to large fresh water river,

(ii) beach backed by no surface water,

(iii) beach backed by estuary,

(iv) beach backed by stagnant lagoon.

The first three locations were in humid climate near the mouth of the Columbia River in Oregon and the fourth location was in semiarid climate near the mouth of the Santa Margarita River at Oceanside, California. The procedure for obtaining these data was to select a line of test holes perpendicular to the shore line. Ground water flow, if any, was assumed to be mainly along such a line. Holes were then dug by hand down to water level and half-pint samples of about half sand and half water were collected. A stadia board was held at the water surface and the elevation recorded. The sand and water were then later analyzed for gradation and salinity, respectively.

Another report of some previous studies of a similar nature has been published (Reference VIII C-4). The original intent of these studies was to find an empirical datum related to tide stage but this objective was not attained. In general, these investigations were carried into the hinterland only 300 to 600 feet. The report presents ground profiles, water table profiles and specific gravity measurements. The specific gravity was used as a salinity index, but as noted previously potable water lies within a very narrow range of specific gravities -- probably too narrow to be apparent in normal field measurements. Nevertheless these data also show that the specific gravity of the ground water drops off sharply to landward.

A third report of water tables in marine beaches (Reference VIII C-1) describes principally some measurements of the profile of the water table transverse to the shore line and observations of the progressive change of the table in response to changing tide level. Changes in salinity with respect to distance from the shoreline were not measured, but the authors concluded that an interchange of water between the ocean and the sand pores takes place. They also concluded that some water rises by capillarity to the sand surface, evaporates, and the sand is cemented into a rigid crust by the residual salts.

Ground water supply for military operations has been discussed in a publication by the Department of the Army (Reference VIII C-11). Chapter 3 (entitled Coastal Ground Water) deals with all sorts of ground water over

large and small coasts. On an amphibious operation however, portions of the coast back some distance from the beach (say several miles) may not be available as sources of ground water. In addition, in an amphibious operation the center of activity is at the beach and at least in early phases, a source of raw water close to the beach is desirable. Nevertheless, since this chapter contains information which does apply to amphibious operations, it is quoted below:

"38. INTRODUCTION. a. General. (1) Coastal ground water is one of the most important sources of supply for human use. It has been heavily exploited, and much information on depth of wells, yields, and quality of ground water has been accumulated. Where military operations along a sea coast require ground water, cities supplied by wells are good sources of information.
(2) Coasts are divided into three classes according to geological conditions affecting ground water: coasts underlain by porous, loose materials such as sand or gravel; coasts underlain by hard rocks with joints and fissures; and coasts underlain by alternate pervious and impervious formations.
b. DEFINITIONS. A shore is the zone over which the water line migrates, or the zone between high and low tides. It includes, however, an additional strip covered by water only during storms. A coast is a broader zone of variable width, extending landward from the shore. The seaward limit of the exposed coast is the coast line.

39. CHIEF PROBLEMS. a. Locating coastal ground water generally is not difficult, but the water quality and yield are affected by many complex conditions. Coastal wells may be affected in two principal ways: they may become salty from encroachment of sea water, and the water level may rise and fall with the tides. The first of these is the more serious problem, for salty water is unfit for most human use and water too salty for human consumption usually is injurious to automotive cooling systems, locomotive boilers, and other types of machinery.
b. The problem of contamination by salt water is affected by several conditions. Among these are the width of the zone in which sea water saturates the formations, the depth at which sea water may occur at various distances from the shore, and the effect of pumping which causes salt water encroachment. The availability of coastal ground water also is affected by stream flow, humidity, seasonal and total rainfall and temperature.
c. The only accurate method of determining the saltiness or degree of the contamination of the water is by chemical analysis. The average concentration of dissolved solids in sea water is about 35,000 parts per million (3.5 percent) and most of the salts are chlorides.

40. KNOWN CONDITIONS IN IMPORTANT COASTAL AREAS. On most coasts, fresh water can be obtained from shallow wells near shore, but deep wells even farther from shore are somewhat salty. This is because the deeper sands which carry salt water are overlain by a zone of fresh water constantly supplied from land sources. On other coasts a porous bed containing fresh water is overlain by a zone of salt water separated from it by an impervious bed.

41. RELATIONS BETWEEN SALT WATER AND FRESH WATER. a. General. (1) Fresh water is lighter than salt water and occupies the upper part of the sand in which both are present, tending to float on the salt water. The position of contact is determined by the head of the fresh water above sea level and by the relative specific gravities of the two liquids. This relationship is shown in figure VIII C-24 (A) and (B).

(2) Where two liquids of different specific gravities fill a U-tube, the heights of the two columns above the point of contact are inversely proportional to the specific gravities.
The same balance of pressure occurs in a small island or peninsula composed of permeable sand. Figure VIII C-24 (B) shows a cross section of a small, sandy island where fresh water is supplied by rainfall. The salt water entirely surrounds the island and extends under it, but the hydrostatic head of the fresh water and the resistance of the pores in the sand prevent the salt water entering the middle zone and mixing with the fresh water. Ordinarily the excess fresh water from the rainfall flows under pressure into the ocean at the shores of the island. In calculations of salt and fresh water levels the average specific gravity of sea water (1.025) is used.
(3) If a sandy island is underlain by an impervious stratum the fresh water is impounded or perched as shown in figure VIII C-24 (C). Most coasts have alternating beds of pervious and impervious material, as indicated in figure VIII C-24 (D). The contacts between salt water and fresh water in the various pervious strata depend upon the hydrostatic pressures of fresh water in each.

b. Effect of Pumping. (1) The amount of fresh water that can be pumped from a coastal formation partly exposed to salt water depends upon the local conditions, the types of well, the rates of pumping, and the rate of recharge of the sand by fresh water. Any decrease in the head of fresh water, either by pumping or by decrease in rainfall, allows a rise in the level of the salt water. The "cone of depression" produced in the fresh-water level around a well allows a corresponding rise in the underlying salt water. If heavy pumping is continued, the well produces salt water. The difference in the specific gravities of salt water (1.025) and fresh water (1.0) is so slight that even a small change in fresh-water head causes large changes in the level of contact. A fresh-water head of 5 feet above sea level is enough to hold back sea water to a depth of 200 feet below sea level.
(2) In developing a water supply along a sea coast, therefore, it is advisable to sink enough wells to yield about twice the required amount of water. Pumping of any one well should be restricted according to draw-down, for, if draw-down is maintained substantially below sea level for extended periods, salt water will enter the well. The pumping rate should not exceed the rate of recharge.
(3) The effect of pumping a well in a sand island is shown in figure VIII C-25 (A) and (B). As the head of fresh water is lowered by pumping, the salt water rises in a point or peak beneath the well. This point might be called a salt-water point of elevation. It is confined to the area of influence of pumping around the well. If it rises to the bottom of the well casing, salt water is pumped.
(4) In an artesian sand, as shown in figure VIII C-25 (C), pumping a well affects much less the line of contact between salt water and fresh water. If pumping does not exceed the rate of intake in the artesian sand the line of contact is not changed. Figure VIII C-25 (C) shows two wells in different artesian sands. One has been pumped at a higher rate than it has been recharged with fresh water, and hence salt water has entered it. The other has been pumped at a lower rate; in it the line of contact between salt and fresh water has not changed.

c. CONTAMINATION IN SHALLOW WELLS. (1) On most coasts, shallow wells ranging in depth from 20 to 50 feet are either dug or driven, and usually end in sand. Contamination can be distinguished roughly in the field by the following scale. The presence of 25 to 100 parts per million of chloride generally indicates a percentage of sea water. Chloride in a proportion of 300 parts per million can be tested readily and indicates definite contamination, but such water can be used for

drinking and, in emergencies, can be used for short periods in automotive cooling systems. Water containing 500 to 1,000 parts per million of chloride is unpalatable. It should not be used in any type of machinery except marine motors.

(2) In shallow wells the zone of diffusion between fresh water and salt water is narrow (less than 100 feet wide) unless affected by heavy pumping. A small sandy island or narrow peninsula, where the area is measured in acres, generally has near the surface a body of fresh water supplied by rainfall (fig. VIII C-25 (A) and (B)). Along shores of large bodies of land which have a permanent water table, fresh water occurs to the edge of the high-water line of the tides, and on sandy beaches fresh water may seep from the beach sand at low tide. In the zone of contamination the water becomes more salty during high tides and freshens during low tides.

d. Contamination in deep wells. (1) In coastal areas underlain by massive, jointed rocks the conditions are different from those in areas underlain by unconsolidated materials. The coast in the vicinity of New Haven, Connecticut, is an example of extensive ground-water development in jointed rocks.

(2) Nearly all the drilled wells in the New Haven coast area produce water from joints. Many of these wells were failures because salt water was encountered at all depths below the water table. The relations between salt and fresh water in the joints and crevices are obscure in many places; one well may yield fresh water while a neighboring one of the same depth produces only salt water. In general, the danger of contamination increases with depth. However, deep wells more than 500 feet from the coast usually do not encounter salt water.

e. Tidal marshes. Tidal marshes are poor sites for wells because the land surface is unfavorable and the water usually is salty both in the marsh and in the rock formations beneath it. The saltiness of the marsh usually is less than that of the ground water, especially in tidal marshes along river mouths where fresh water is discharged continually.

f. Bars and spits. Most bars, spits, and barrier beaches are unfavorable locations for wells because the ground water in them is somewhat salty. The sand composing such features is extremely porous, and high waves carry salt water over large parts of small bars, saturating them with sea water. Large barrier beaches such as that of the western Gulf Coast, however, have enough area to maintain shallow fresh-water wells.

g. Effects of joints in bedrock. In a body of jointed and fractured rocks along a coast, sea water enters the rocks through joints and may penetrate some distance inland. This penetration varies with the pressure relationship between fresh water and salt water and the line of contact is typically irregular. The extent of penetration by sea water can be determined only by consulting existing well records or by test drilling.

h. Effect of recent silt deposits. At a number of other places along the Atlantic Coast, deposits of recent mud have been lain down by streams flowing into the sea. This has provided an impervious cover over porous sands in the estuaries. The sands beneath this cover contain fresh water under low head from nearby land areas, and can be tapped by shallow wells. This condition exists in many harbors.

i. Other influences such as tides and temperatures. Fluctuations of water levels with the tides have been recorded in wells along many seacoasts. In wells near shore the fluctuations may range from a few inches to several feet. Wells in fractured rock formations generally are affected to about the same extent as wells in pervious sands. Higher temperature increases evaporation and accelerates seepage and percolation. A seasonal rise in ground-water level from heavy rainfall may increase the head of fresh water and thus cause seasonal increases in fresh-water yield."

The foregoing data suggest that the zone of diffusion at the water table surface between water coming from the sea and water coming from the land is relatively narrow. This zone cannot properly be called an interface between salt water and fresh water since the water may be salty on both sides of the interface. Probably on most sand beaches the ground water will have some salinity near the shore, either from a recent sea water intrusion or inundation, or spray from storm breakers. These data seem to indicate that the major zone of diffusion is relatively narrow with a "fringe" zone on the land water side which is much wider, but between lower saline limits.

(4) Conclusions: On the basis of these studies some tentative conclusions can be advanced which will be of some assistance in developing sources of raw water on ocean beaches:

(i) Water behind most ocean beaches is less saline than sea water and consequently is a potential source for cooling water or potable water. In many cases it is a better source of raw water for distillation units than the ocean.

(ii) Surface water in humid climate which is behind a beach and is subject to tidal variations is apt to be more saline than the surrounding ground water.

(iii) Surface water and ground water in semiarid climate are not apt to be potable in shallow wells immediately behind a beach especially if the hinterland is well developed agriculturally and irrigated by water from local wells.

(iv) Ground water in humid climate is apt to be less saline behind a beach if tidal lagoons or estuaries are not landward of the ground water (e.g. if the beach is not a bay-mouth bar).

(v) A shallow well at the bottom of a low topographic depression behind a beach is less apt to yield potable water than a site on the slope leading into it since sea water may have once evaporated here leaving residual salts in the basin.

(vi) The major diffusion between sea water and land water appears to be in a narrow zone, so that if ground water on a beach is potable it should be found close to the beach.

6. References:

VIII C-1: Emory, K. O. and Foster, J. F. -"Water Tables in Marine Beaches" - Sears Foundation: Journal of Marine Research, Vol. VII, No. 3, 15 November 1948, pp 644-654.

VIII C-2: Gibson, George E. -"Effect of Oil on Ocean Waves" - U. S. Navy Contract NPS 385 Woods Hole Oceanographic Institution, 26 October 1945, (unpublished).

VIII C-3: Glenn, A. H. - "Progress Report on Solution of Wave, Tide, Current and Hurricane Problems in Coastal Operations" - Oil and Gas Journal, June, 1950.

VIII C-4: Isaacs, J. D. and Bascom, W. N. - "Water Table Elevations in Some Pacific Ocean Beaches" - Transactions, American Geophysical Union, Vol. 30, No. 2, April 1949, pp 293-294.

VIII C-5: Iseley, J. A. and Crowl, P. A. - "The U. S. Marines and Amphibious War: Its Theory, and its Practice in the Pacific" - Princeton University Press, Princeton, N. J., 1951.

VIII C-6: Marshall, Colonel S. L. A. - "The Soldier's Load and the Mobility of a Nation" - The Combat Forces Press, Washington 6, D. C., 1950.

VIII C-7: O'Brien, M. P. - "Report on Surf Characteristics of Landing Craft" - Institute of Engineering Research University of California, Series 3, Issue 104, Berkeley, California, 11 May 1945, (unpublished).

VIII C-8: Patrick, Lt. D. A. - "Ground Water Adjacent to Four Pacific Ocean Beaches" - Institute of Engineering Research, University of California, Series 29, Issue 35, Berkeley, California, 21 September 1950.

VIII C-9: U. S. Army Air Corps -"Arctic Manual", 1940.

VIII C-10: U. S. Army - "Water Supply and Purification" - War Department Technical Manual 5-295, 1942.

VIII C-11: U. S. Army - War Department Technical Manual 5-296, 1 February 1944.

VIII C-12: U. S. Marine Corps - "Terrain, Hydrography, and Weather" - Amphibious Manual, AM - 2, 1951, RESTRICTED.

VIII C-13: U. S. Navy - "Wind Waves at Sea: Breakers and Surf" - Hydrographic Office Publication No. 602, 1947.



FIG. VIIIC-1 - Offloading LCVPs. Note numbers on side of ship indicating debarkation stations.



FIG. VIIIC-2 - Offloading equipment into an LSU.



FIG VIIIC-3 - Stern view of an LSD.



FIG. VIIIC-4 - Refueling operations.



FIG VIIIC-5 - LCVP being lowered from on APA using the portside boat davits. Roil-looded boats of this noture moy be looded prior to lowering. If the craft is loaded, crewmen should toke their weight out of the boot by supporting their weight on the grob lines rigged to the strong bock on the boot dovit. Some of the grob lines may be seen dongling from the dovits. In on Amphibious Londing Operotion the roil-looded boots usuolly corry vitol corgo which is needed in support of the first ossoult woves.



FIG. VIIIC-6 - An LST listing to storboard preparatory to launching on LSU. Note fenders which have been rigged to protect the hull of the LST.



FIG VIIIC-7 - DUKW entering on LST. The obove photo indicates an incorrect method of boording an LST. The general procedure is to drive in bow first and back out stern first when debarking. Boarding ond deborking of amphibious craft from the bow doors of londing ships requires close cooperation between the ships crew ond the vehicle operotor. A domoged DUKW which is unable to board the ship through the bow doors may be salvaged by hoisting in the boot dovits, if proper proceedures are used.



FIG. VIIIC-8 - Employment of DUKW for transporting cargo.



FIG. VIIIC-9 - Troops debarking from a transport into landing craft utilizing the conventional debarkation net. Note heaving and pitching of the first and second boats as they ride on the crest of a small wave. The debarking of troops from landing ships into landing boats, when required, follows the same general procedures as debarking from transports.



FIG. VIIIC-10 - Offloading cargo and equipment into landing craft. Incorrect rigging has caused the jeep being unloaded to slip and turn on its side. Caution and proper unloading techniques must be observed in unloading of heavy equipment into landing craft and lighters. A heavy piece of equipment may seriously damage the landing craft unless properly lowered. The person in charge of unloading must time the lowering of equipment into the boat with the rise and fall of the boat on the waves and swell.



FIG. VIIIC-11 - An illustration of the correct procedures for debarking of troops into landing craft. The debarkation net forms a direct ladder into the bottom of the boat. Note men at the corners of the net keeping it stretched taut for descending personnel. Personnel are using vertical strands of the net for handholds, as is proper. Note one of the tending lines attached to the lower corner of the net for raising and lowering the bottom of the net prior to the arrival and deporture of boats. Equipment carried by troops should be arranged so that it may easily be removed in event the man should fall into the water. Vertical and horizontal distances should be maintained between troops while descending the net to prevent bumping into each other, and entangling equipment. Rifles and other weapons should be unslung from the shoulder after entering the boat and so placed that a man falling from the net will not be endangered by them.



FIG. VIIIC-12 - Lowering an LCM from an AKA. Frapping lines are attached to the boat and tended on deck to keep the craft from swinging and damaging the ship's hull. Unloading of boats in a Debarkation Operation is generally accomplished simultaneously from both sides of the ship, weather conditions permitting. If sea conditions are such to require a lee, portside boats are usually lowered first. The boat is usually lowered to within a few feet of the surface of the water and then, during a period of relative calmness, such as occuring during a period between waves, is lowered into the water.

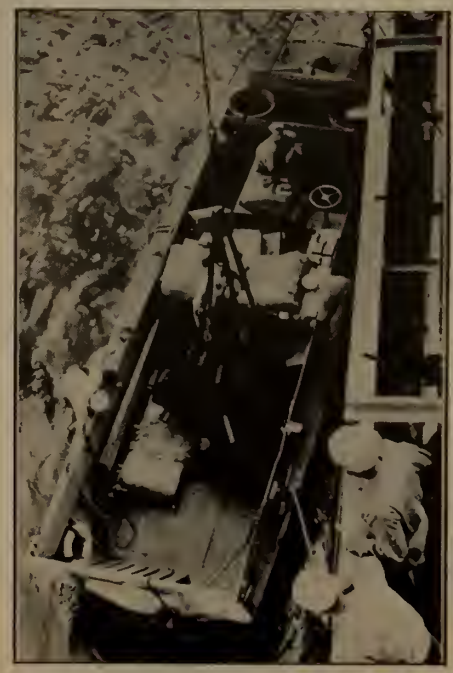


FIG VIIIC-13 - An LCVP being used for cargo unloading.



RESTRICTED Security Information





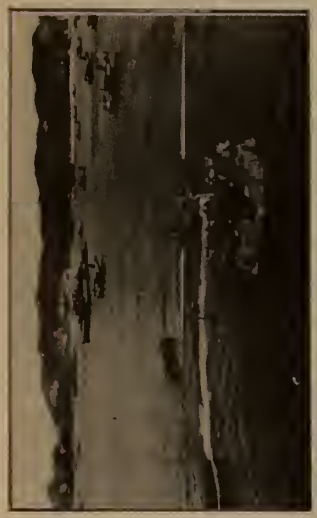


FIG. XIIIC-14 - A pontoon barge being used for transporting aircraft. Barges of this sort are used extensively in unloading cargo and equipment from support ships. They are also used as floating cargo and fuel dumps. A floating fuel dump may be established on a pontoon barge to refuel landing craft and amphibious vehicles during the debarkation and assault operations.



FIG. XIIIC-15 - LVTs debarking from an LSD. The general proceedure for debarking amphibious vehicles from an LSD is to flood the well deck and allow the vehicle to proceed forward, bow first, under its own power. One of the primary uses of the LSD is to transport tanks which are carried in tank lighters (such as an LSU). Debarkation of the tank lighters is also accomplished by flooding.

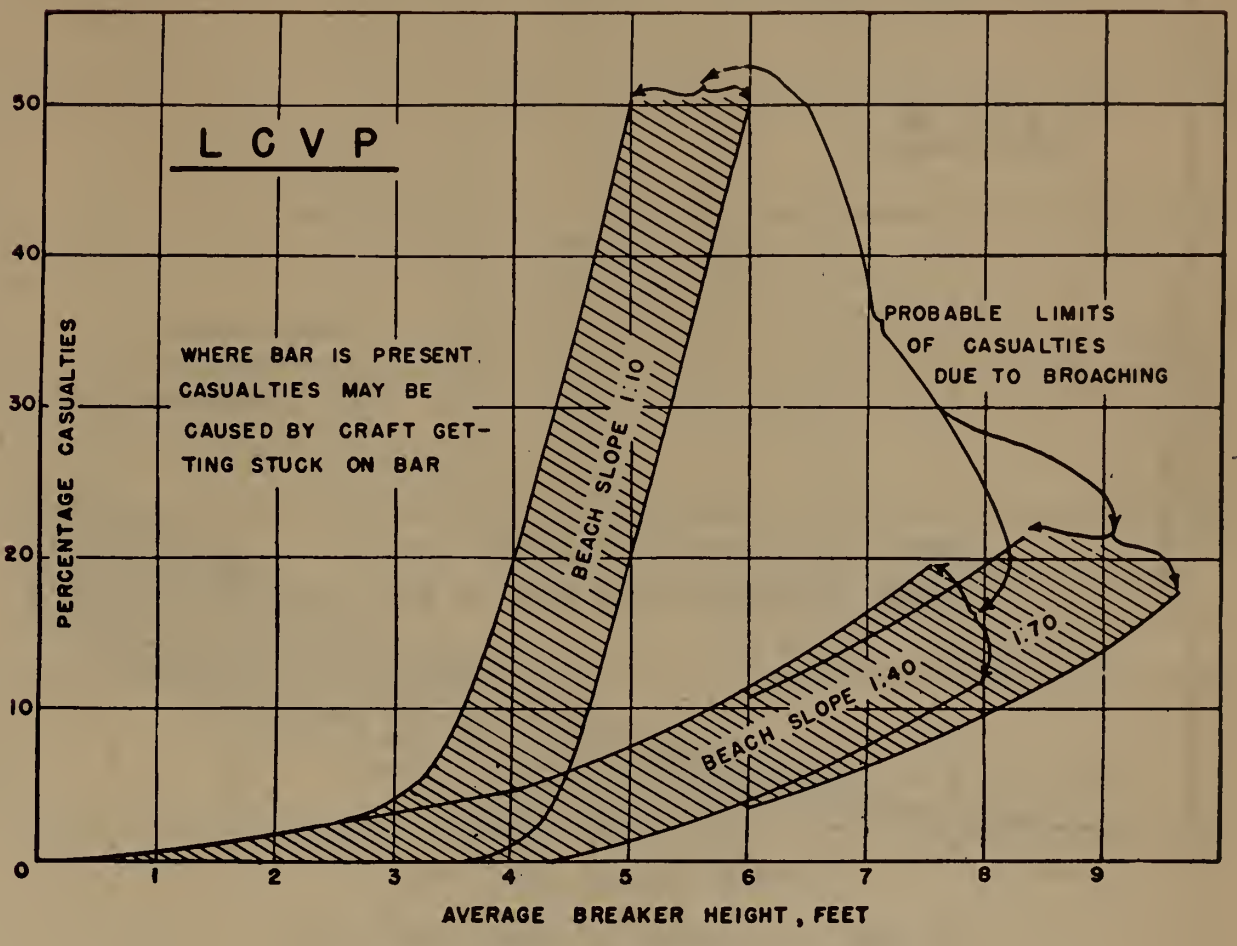



FIG. VIII C-16a - GRAPH OF CASUALTIES, LCVP

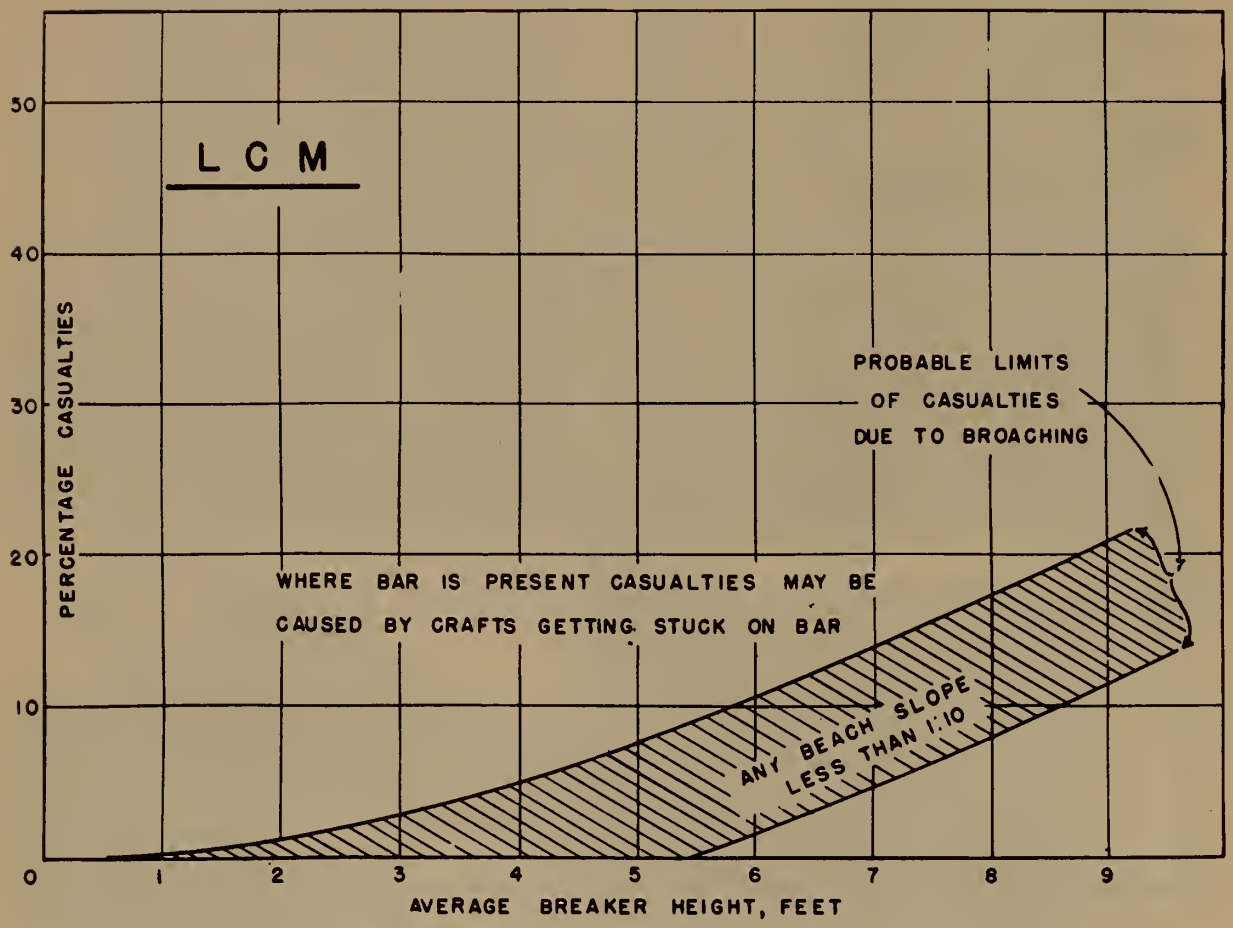



FIG. VIIIC-16b - GRAPH OF CASUALTIES, LCM

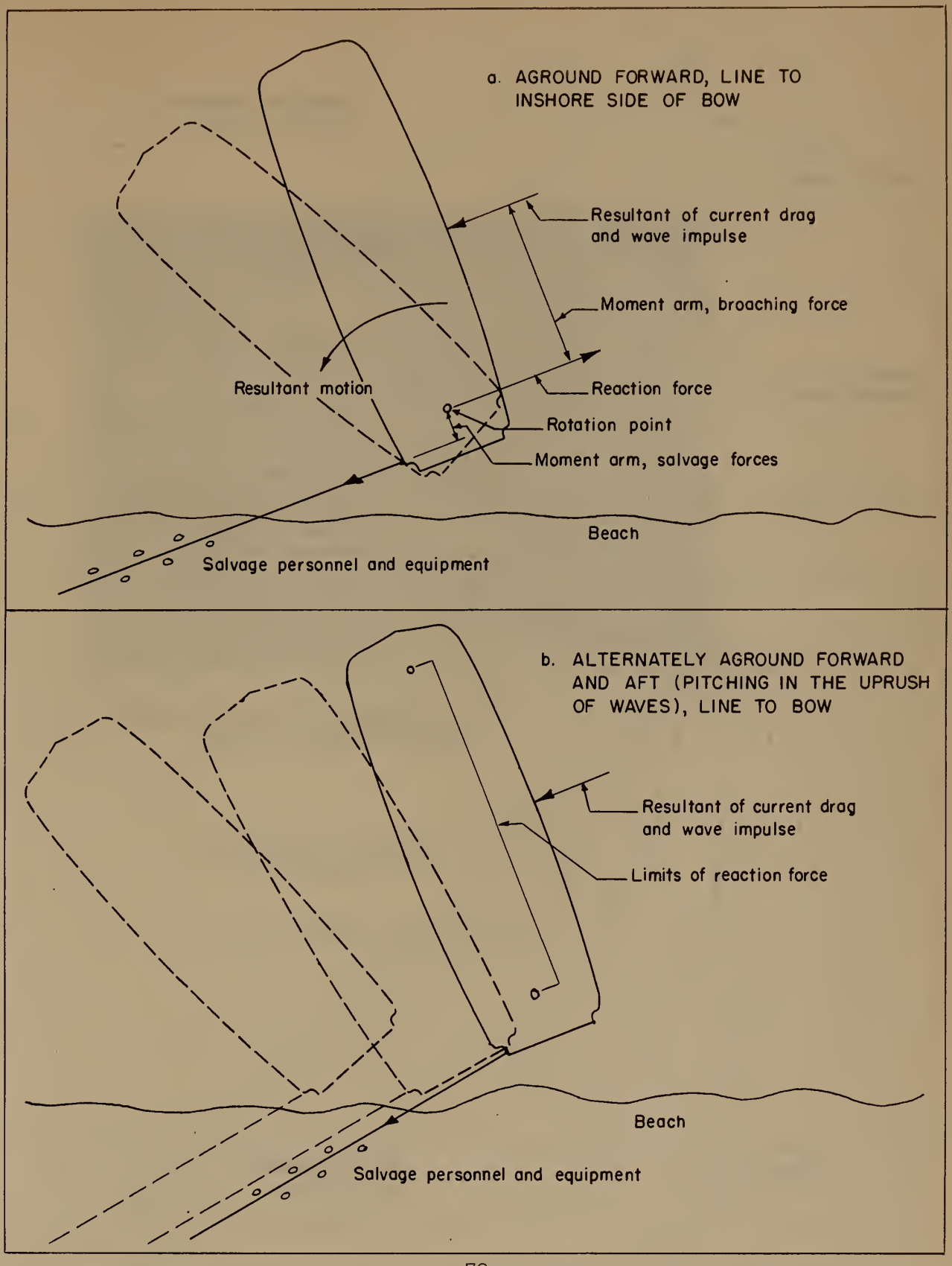





FIG. VIII C-17 - FORCES ACTING ON LCVP WHEN AGROUND

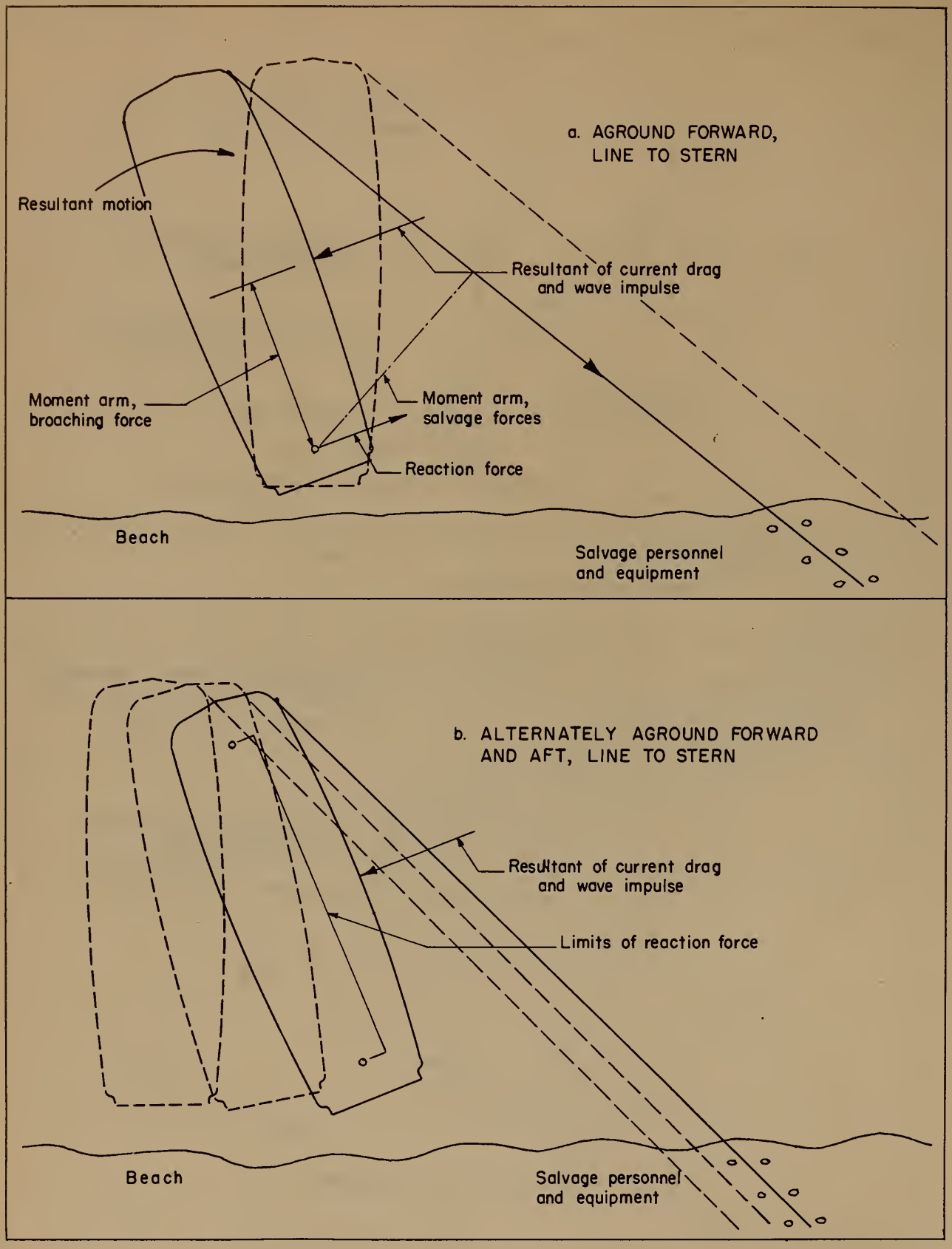
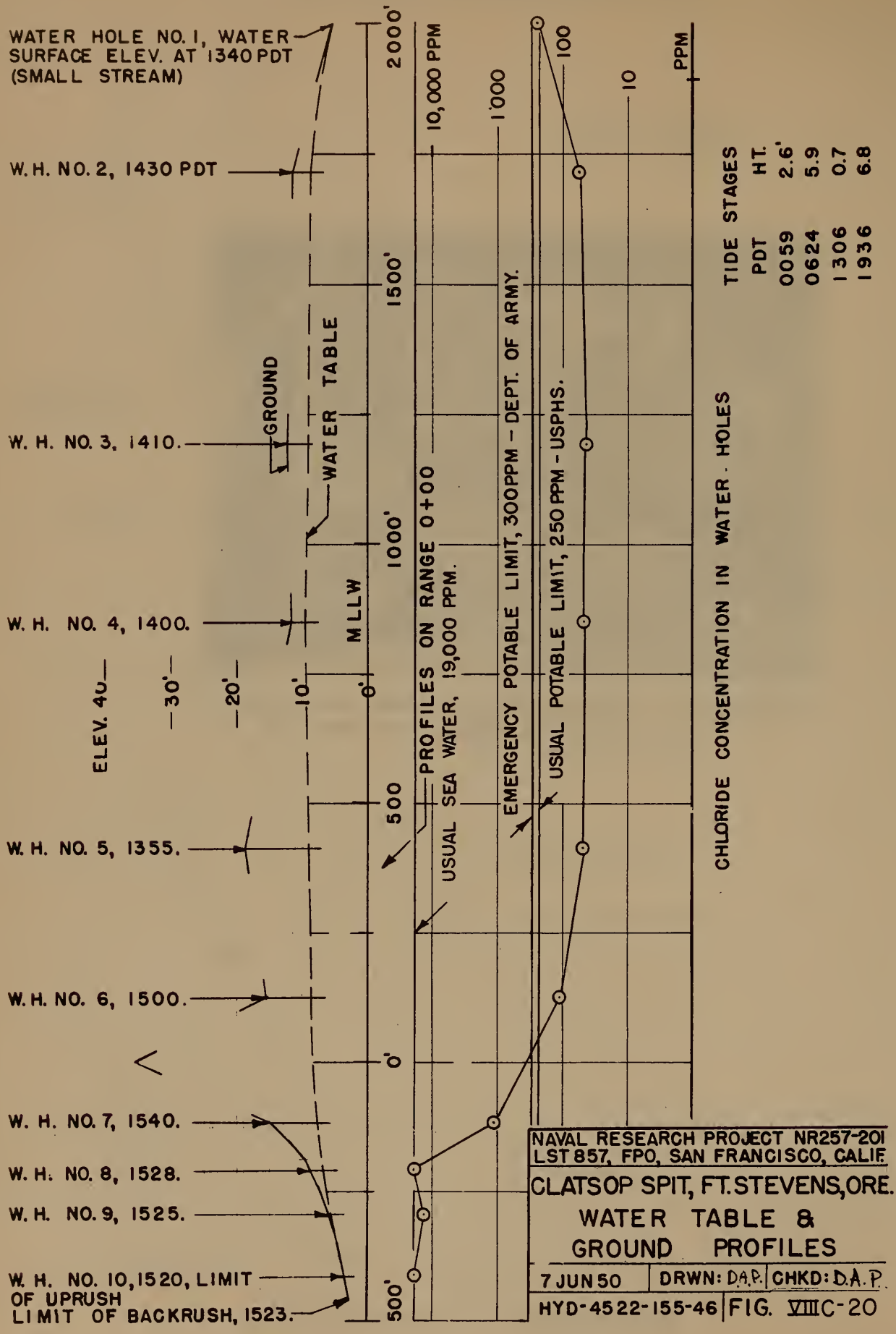
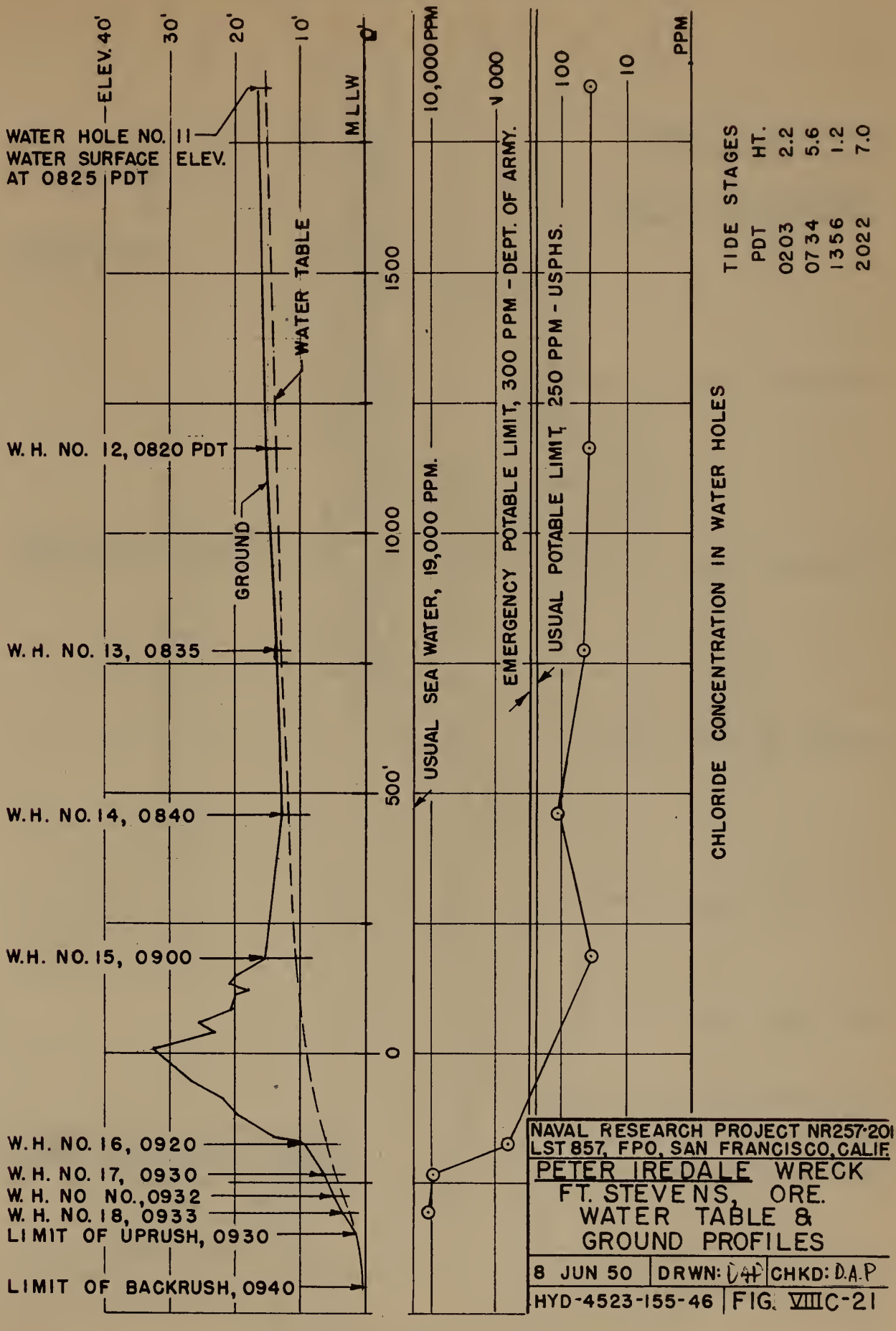


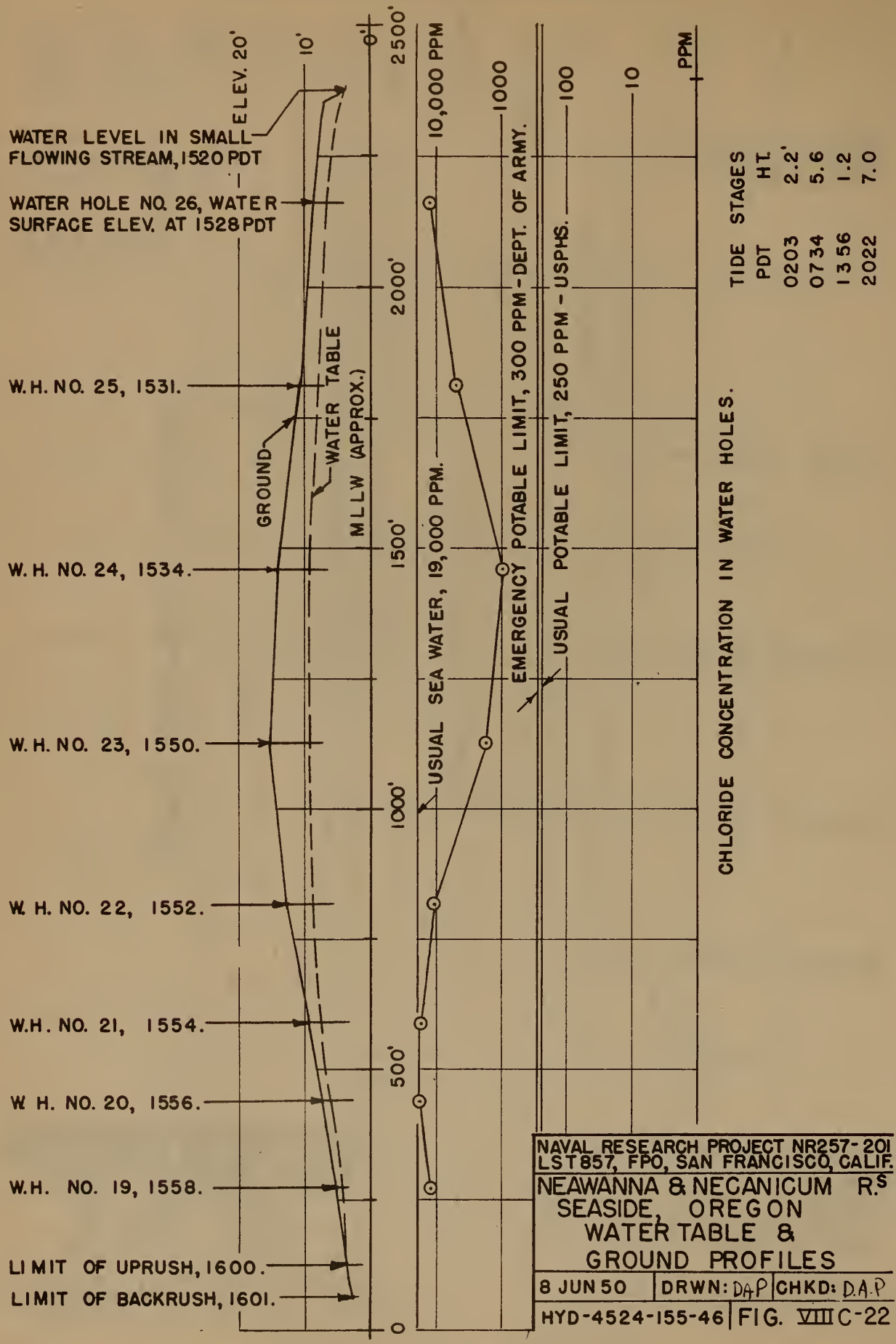

FIG. VIII C-18 - FORCES ACTING ON LCVP WHEN AGROUND



FIG. VIII C-19 - Marines manually unloaded water drums in the Peleliu invasion, September 1944


WATER HOLE NO. 1, WATER SURFACE ELEV. AT 1340 PDT (SMALL STREAM)
W. H. NO. 2, 1430 PDT
W. H. NO. 3, 1410.
W. H. NO. 4, 1400.
W. H. NO. 5, 1355.
W. H. NO. 6, 1500.
W. H. NO. 7, 1540.
W. H. NO. 8, 1528.
W. H. NO. 9, 1525.
W. H. NO. 10, 1520, LIMIT OF UPRUSH
LIMIT OF BACKRUSH, 1523.
GROUND
WATER TABLE
MLLW
ELEV. 40
30'
20'
10'
0'
2000'
1500'
1000'
500
0'
500'
PROFILES ON RANGE 0+00
USUAL SEA WATER, 19,000 PPM.
10,000 PPM
1'000
100
10
PPM
EMERGENCY POTABLE LIMIT, 300PPM – DEPT. OF ARMY.
USUAL POTABLE LIMIT, 250 PPM - USPHS.
CHLORIDE CONCENTRATION IN WATER HOLES
TIDE STAGES
PDT HT.
0059 2.6'
0624 5.9
1306 0.7
1936 6.8
NAVAL RESEARCH PROJECT NR257-201
LST 857, FPO, SAN FRANCISCO, CALIF.
CLATSOP SPIT, FT. STEVENS, ORE.
WATER TABLE & GROUND PROFILES
7 JUN 50 | DRWN: D.A.P. | CHKD: D.A.P.
HYD-4522-155-46 | FIG. VIIIC-20

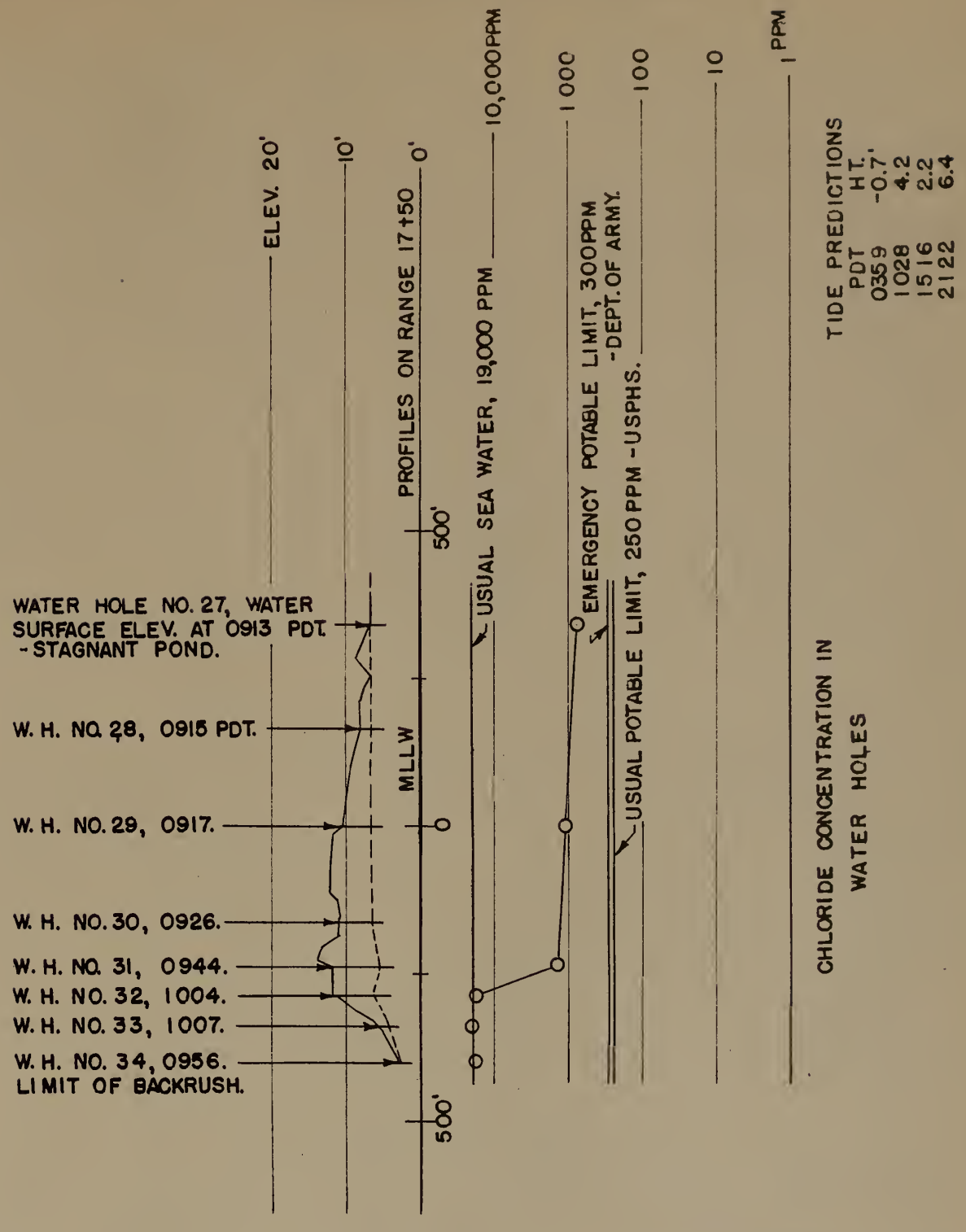

ELEV. 40'
30'
20'
10'
0'
MLLW
WATER HOLE NO. 11
WATER SURFACE ELEV. AT 0825 PDT
WATER TABLE
GROUND
W. H. NO. 12, 0820 PDT
W. H. NO. 13, 0835
W.H. NO. 14, 0840
W.H. NO. 15, 0900
W.H. NO. 16, 0920
W. H. NO. 17, 0930
W. H. NO NO., 0932
W. H. NO. 18, 0933
LIMIT OF UPRUSH, 0930
LIMIT OF BACKRUSH, 0940
1500
1000
500'
0
10,000 PPM
1000
100
10
PPM
USUAL SEA WATER, 19,000 PPM.
EMERGENCY POTABLE LIMIT, 300 PPM - DEPT. OF ARMY.
USUAL POTABLE LIMIT, 250 PPM - USPHS.
CHLORIDE CONCENTRATION IN WATER HOLES
TIDE STAGES
PDT HT.
0203 2.2
0734 5.6
1356 1.2
2022 7.0
NAVAL RESEARCH PROJECT NR257-201
LST 857, FPO, SAN FRANCISCO, CALIF.
PETER IREDALE WRECK
FT. STEVENS, ORE.
WATER TABLE & GROUND PROFILES
8 JUN 50
DRWN: DAP
CHKD: D.A.P
HYD-4523-155-46
FIG. VIIIC-21




NAVAL RESEARCH PROJECT NR257-201
LST 857, FPO, SAN FRANCISCO, CALIF.

NEAWANNA & NECANICUM R.S
SEASIDE, OREGON
WATER TABLE &
GROUND PROFILES

| 8 JUN 50 | DRWN: DAP | CHKD: D.A.P |
|---|---|---|
| HYD-4524-155-46 | FIG. VIII C-22 | |




| NAVAL RESEARCH PROJECT NR257-201 CAMP PENDLETON, OCEANSIDE, CALIF. | | |
|---|---|---|
| SANTA MARGARITA RIVER WATER TABLE & GROUND PROFILES | | |
| 17 JUL 50 | DRWN: D.A.P. | CHKD: D.A.P. |
| HYD-4547-155-56 | FIG. VIIIC-23 | |

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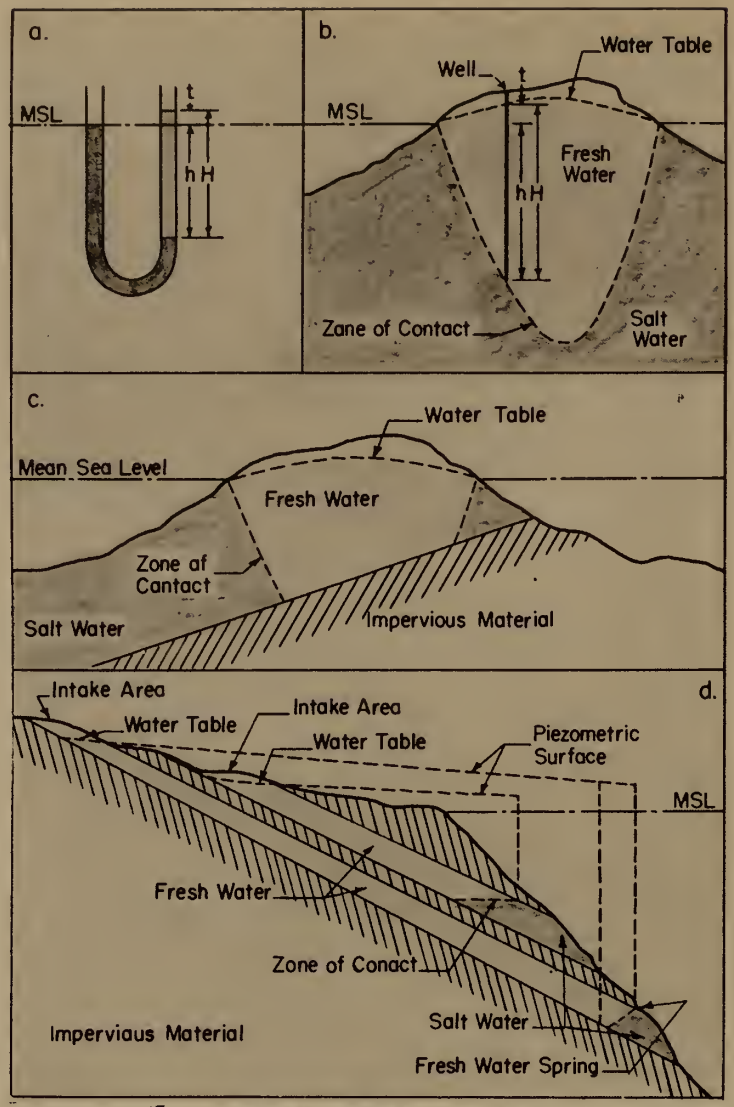



FIG. VIIIC-24 - Relation between fresh water and salt water in water-bearing sands when not disturbed by pumping. (a) U-tube showing balance between liquids of different specific gravities; (b) section through an island composed entirely of sand and surrounded by salt water; (c) section through a sand island underlain by impervious material; (d) section through a coastal plain composed of alternate layers of pervious and imperious materials.

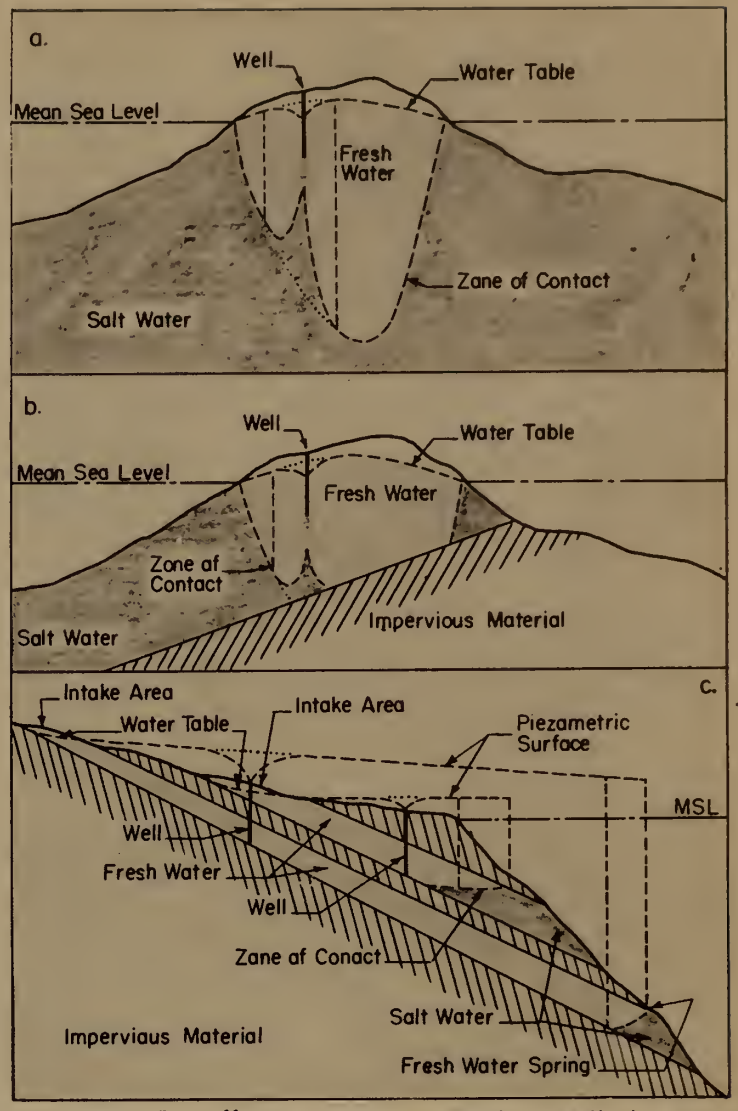



FIG. VIIIC-25 - Effect of pumping water from wells in sand exposed to salt water contamination: (a) in an island composed of sand, (b) in a sand island underlain by impervious material, (c) on the coast of a large land area where artisian sands are confined between impermeable strata.

(Courtesy, U.S. Army)

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# MANUAL

# OF

# AMPHIBIOUS OCEANOGRAPHY

## SECTION IX

## INVASION HARBORS

BY

J. R. MORISON



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# MANUAL OF AMPHIBIOUS OCEANOGRAPHY

## SECTION IX: INVASION HARBORS

BY

J. R. MORISON

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# IX INVASION HARBORS

## Abstract

The supply of an invasion force requires various forms of harbors in friendly and enemy territory. Harbor selection, planning and construction for temporary and permanent supply lines are presented here. Harbor and shoreline structures are discussed in regard to types, their uses and methods of construction. Successful continuation of supply to an invasion force depends upon the improvement and protection of temporary harbors, and the repair and operation of captured permanent harbors.

## Acknowledgement:

The illustrations used in this section of the manual were obtained from many sources. The number in parentheses at the end of each figure caption refers to the source as listed in the bibliography.

# MANUAL OF AMPHIBIOUS OCEANOGRAPHY

## SECTION IX: INVASION HARBORS

### A: INTRODUCTION

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## A. INTRODUCTION

One of the first amphibious landings of any size was made by the Persian military genius, Xerxes I. He used boats to construct a floating bridge across the Hellespont, and his troops crossed this bridge to invade Europe 400 years before the birth of Christ. Some 2,350 years later, the allied invasion of Northern France (Normandy) was undoubtedly the largest amphibious operation in the history of the world. An even greater feat than the invasion itself was the planning and construction of five artificial harbors - two of them relatively large harbors (Figure IX A-1 shows the artificial harbors at Arromanches, France.)

The purpose of this summary is to briefly describe and discuss invasion harbors and all important related subjects. In an invasion, there are three general phases of supply operations. All three require considerable planning and organization if a successful supply line is to be maintained.

The first phase is that of supplying the initial assault. In this phase, supplies are:

(i) carried ashore by troops and landing craft (Figures IX A-2, 3, 4, 5 and 6)

(ii) discharged from beached ships (Figure IX A-7)

(iii) discharged from grounded ships - to which has been constructed a dirt pier (Figures IX A-8 and 9), or

(iv) discharged from grounded ships or moored ships at a pontoon causeway or discharged from ships to ferries and subsequently unloaded over some temporary pier (Figures IX A-10, 11, 12, 13, 14 and 15.)

The second phase in invasion supply may occur simultaneously with the first. It should include construction of a temporary harbor to protect the mooring sites and piers; as well as improvement and expansion of the piers. However, this phase sometimes consists only of improving the piers without adding any protection such as breakwaters. When this is the case, the success of the supply line may depend on a narrow margin of continuous good weather. Figures IX A-16, 17, 18 and 19 show the temporary harbor on the Normandy Coast with its piers.

Permanent harbors are of course most useful for supplying invasion forces. The third phase is to repair, operate and improve captured harbors as permanent harbors of supply; as well as to build permanent harbors. This work requires a special knowledge of harbor construction, and is generally performed by special crews -- not by the assault-trained crews that do the work of the first and second phases. Figures IX A-20, 21 and 22 show some of the more permanent types of harbor construction. Figures IX A-23, 24 and 25 show harbor damage caused by bombing and enemy action.

The rehabilitation of a harbor includes:

(i) clearance of mines and enemy demolitions

(ii) salvage of sunken ships

(iii) repair of piers, breakwaters, lockgates, and their machinery, water supply equipment etc.

(iv) repair of roads, railroads and docks,

(v) dredging

(vi) repair of cranes and similar machinery, of workshops - and supplying them with necessary power, and

(vii) clearing and repairing inland waterways.

The general construction problems are covered in the later sections on piers, breakwaters, jetties, etc. Figure IX A-26 and 27 show the method of repair of several quays. It should be noted that the Port of Lyantez North Africa was captured undamaged but it had to be cleared of sunken ships.

This section will not describe in detail the first phase of invasion supply. For a complete picture of the second and third phases, it is necessary to include the problems of the choice of a harbor site; the general features of a harbor; the effect of waves; and the construction of temporary and permanent piers, breakwaters, jetties, seawalls and groins. Many of the factors mentioned briefly here are covered more fully in other sections of the Manual of Amphibious Oceanography.



FIGURE IX A-1 - The Mulberry "B" harbor at Arromanches (80)



FIG. IX A-2 - Landing craft bringing men and supplies ashore on the Normandy beach (44)



FIG. IX A-4 - Troops and equipment come ashore onto onto the black volcanic sand of Attu (1)



FIG. IX A-6 - Bulldozer pulls supply trailer off landing boat, North Africa (70)



FIG. IX A-3 - Amphibious operation, Cape Gloucester (34)



FIG. IX A-5 - American troops unload rations from landing craft, Guadalcanal (70)



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FIG. IX A-7 - Unloading operations at Iwo Jima (56)



FIG. IX A-9 - Unloading directly to Athey trailer in the Aleutians (32)






FIG. IX A-8 - Tractors pulling heavy loads around on a beachead at Green Islands, Feb 1944 (88)



FIG. IX A-10 - LSTs disgorge their cargo of tanks and supplies over pontoon causeway at Licata, Sicily (15)

FIG. IX A-11 - Navy Seebees setting up pontoon pier for landing men and equipment at Inchon, Korea (57)



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FIG. IX A-12


FIG. IX A-14


FIG. IX A-16


FIG. IX A-13


FIG. IX A-15

FIG. IX A-12 - Completed approach to old civilian jetty, Green Beach, Parang (35)

FIG. IX A-13 - Seebee-built pontoon pier at Wolmi Do, Korea, for unloading troops and supplies (14)

FIG. IX A-14 - LST loading mechanized material onto a Rhino ferry for delivery on the French coast (44)

FIG. IX A-15 - Rhino discharging cargo directly over beach, near ramp of wrecked LCI (53)

FIG. IX A-16 - Artificial harbor at Arromanches, France (53)



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FIG. IXA-17 - LCT discharging troops over poontoon causeway (53)



FIG. IXA-18 - Rhino ferry discharging cargo over pontoon causeway (53)



FIG. IXA-19 - Floating roadways (whale) (53)



FIG. IXA-20 - Temporary piers, Guam (40)



FIG. IXA-21 - American transport unloading supplies at pontoon finger piers constructed by Seebees (15)



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FIG. IXA-22 - General view of the boat basin, Peleliu (21)



FIG. IXA-23 - The damage at the port of Le Harve, France (59)



FIG. IXA-24 - View along the Naples waterfront (29)



FIG. IXA-25 - Sunken ships and wreckage of port facilities of Naples (29)



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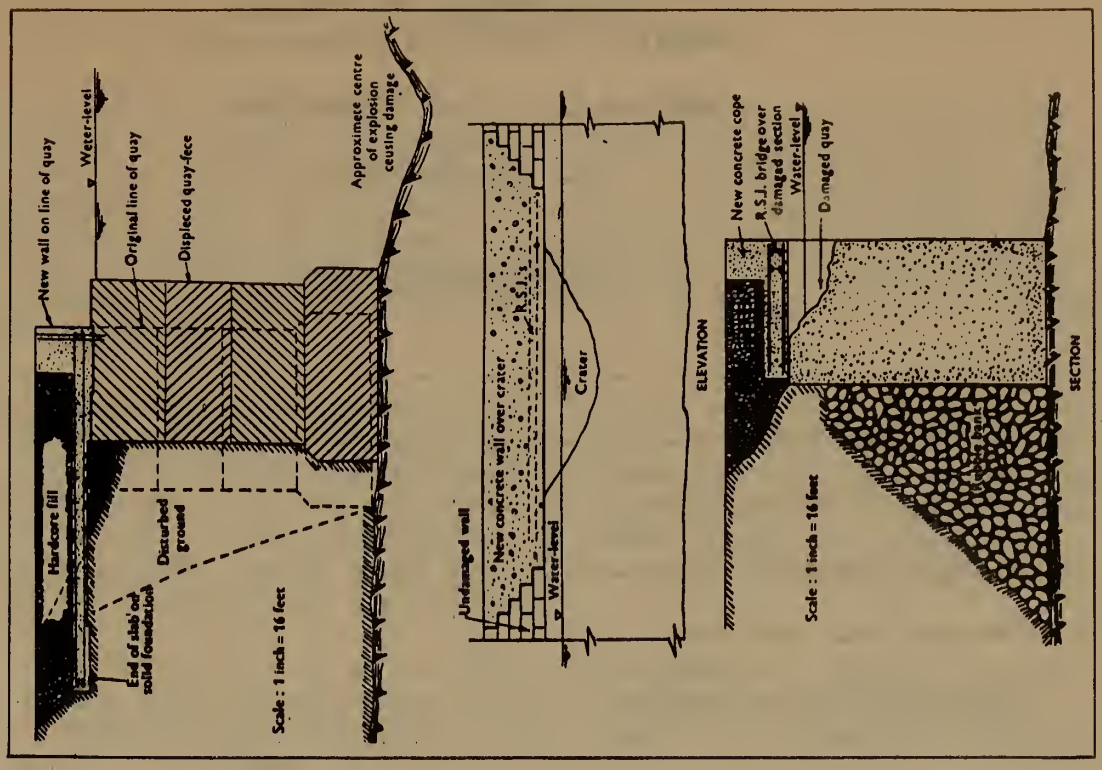



Fig. IX A-27 - Typical quay repair (55)

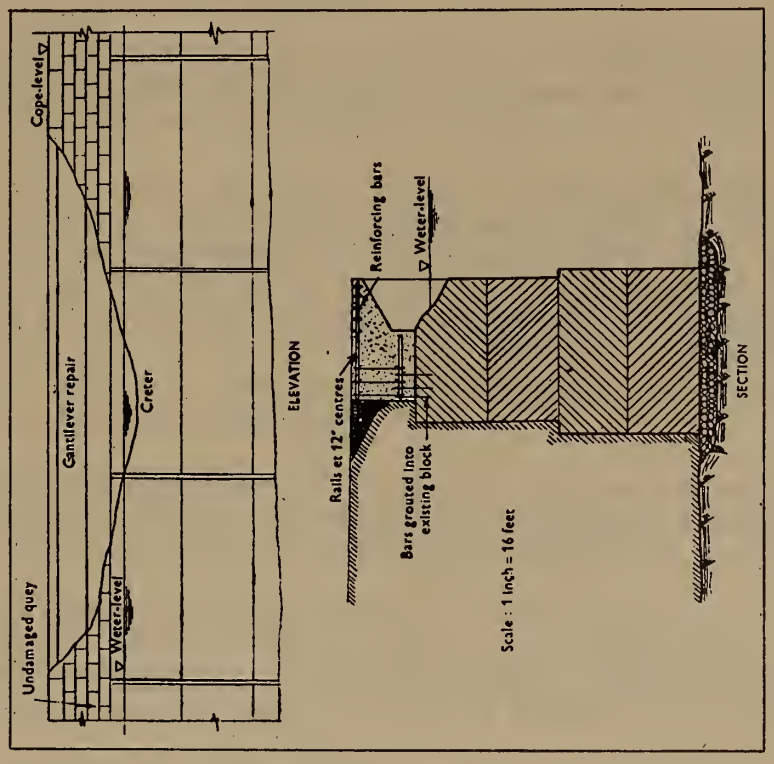



Fig. IX A-26 - Typical quay repair (55)

SECTION IX: INVASION HARBORS

B. HARBOR SITE

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## B. HARBOR SITES

A harbor site is usually chosen before any study of waves and currents is made. The choice of a harbor site depends primarily on its usefulness. For military operations, the tactical situation must be considered as well as the physical properties of the general harbor location. An invasion harbor has somewhat different initial requirements than a supply harbor. Figure IX B-1 shows that each invasion harbor of Normandy was opposite a population center where road nets were available, and also that the harbor of Cherbourg was nearby for future supply activity.

In choosing the site for a harbor, full utilization should be made of existing (man-made) harbors and also of natural harbors such as bays, lagoons, rivers, and inland waterways. Figures IX B-2 and 3 are examples of a natural reef harbor and a river harbor that has been improved. If possible, a harbor should have more than one entrance. The condition of the bottom should be investigated, and if found poor, then mooring buoys or similar structures should be provided. (Notice buoys in Figures IX A-1 and IX F-11).

Where a military harbor is to be built, the exposure to tides, to wave action, and to the adverse refraction of waves are the most important considerations in addition to the tactical uses. Some of the important tactical considerations are as follows:

(i) If the harbor is to be built on the shoreline of a hostile country, the distance that building materials will have to be transported must be considered.

(ii) The useful life of the harbor must be determined, since this will effect the type of construction. Temporary harbors cannot be expected to withstand many severe storms.

(iii) The harbor must be large enough to accommodate the necessary number of ships. This means a study must be made of bottom slopes since deep water increases the construction problems. Possible erosion of the foundation of protective structures and the filling or silting of the harbor should be carefully investigated.

(iv) Where amphibious landings are to be made, the trafficability of the site must be considered.

(v) Another important factor in harbor site determination is the length of time that will be required to construct a harbor that will give the necessary protection from wave action and currents.

(vi) In military operations, the cost of construction is of secondary importance, as shown by Table IX B-I. This table shows that the approximate average cost of a base including harbor, airfield, roads, etc. was about two and one-half million dollars per month.

(vii) The conservation of materials may be extremely important, and in addition, artificial harbor building may be limited by the lack of civilian production facilities.

Although the primary purpose of this summary is to consider military harbors, it is also important to understand the reasons for the choice of civilian harbors, since their location is generally involved in strategic operations. The choice of harbor sites for civilian use is determined by the cost of construction and the ability of the local interests to pay or justify the cost. Political interests may also play a role in harbor locations. In general, harbors are built for commercial, refuge, fishing, recreation, and national defense purposes or a combination of these purposes.

1. Commercial Harbors: are located where large population centers are or can exist with plans that include both city and harbor expansion. Accessibility to railroads and highways is both an initial and future consideration. The amount of space needed for the expected trading ships and warehouses must be computed. The amount of import and export from local manufacture and the size of the customer markets must be estimated. The available labor markets for the maritime industry must be considered. The possibility of the re-location of railroads, highways and bridges as a result of the harbor must also be considered.

Some physical considerations of a harbor site are as follows:

(i) First and most important is that natural harbors - such as bays - should be fully utilized.

(ii) The source of building materials must be considered - both the distance they are from the harbor, and available methods of transporting them.

(iii) A knowledge of the bottom conditions and bottom slope is necessary, with particular reference to the size of the harbor and the depth of water in which construction is to take place.

(iv) The exposure to wave action must be considered.

(v) The effect of harbor construction on the natural movement of sand along the shoreline and the movement of bed-load material from rivers should be studied.

(vi) The size of a harbor on an open coast, that is constructed artificially by breakwaters, may be limited by the size of the wave that can be generated in the limited fetch of the harbor itself. For instance, in the case of the natural harbor of San Francisco Bay, it is necessary to have breakwaters within the bay to protect the shipping facilities from the waves generated within the bay.

In addition to commercial harbors, there are harbors of refuge, fishing harbors, recreational harbors, and national defense harbors. The location of these harbors may make an ideal place for a supply harbor for an invasion force.

2. Harbors of Refuge: are constructed in remote sections of the coast where other types of harbors are spaced far apart and where storms are severe. They are often combined with fishing harbors. The harbor must have a safe

entrance during severe storms. The size of ship to be accommodated is an important factor. Otherwise the same physical consideration as for commercial harbors must be considered.

3. Fishing Harbors: are located as close as possible to the fishing grounds (15 to 20 miles) and to the customer market. Modern fishing methods and quick freezing have extended the distance between fishing ground, fishing harbor and market. Some other factors to be considered are the labor market for fishermen and cannery workers, the housing areas available for the fishermen, and fishermen supplies and services. Since fishing boats are generally small as compared to commercial boats, the greatest protection from wave action must be obtained for the lowest possible cost.

4. Recreational Harbors: are generally combined with, or are very near, commercial harbors. Recreational harbors include yacht harbors. Although swimming may be provided for, it is not a consideration for the site of a harbor. The population center or the amount of contribution from surrounding centers is the main consideration for recreational harbors such as those on the New Jersey coast or San Francisco Yacht Harbor.

5. National Defence Harbors: Harbors created for the national defense are generally part of, or near to, commercial harbors. Examples of this combination are Mare Island in San Francisco Bay and the harbor at San Diego. Some military harbors are built as permanent outposts such as in Alaska and on Guam. In these cases, the tactical situation and physical features must be balanced in the choice of a site.

4.

TABLE IX B-I*

Some Naval Bases Constructed and Operated
During World War II

| Location of Base | Cost of Base | Approximate Dates |
|---|---|---|
| Guam | $280,795,700 | July 1944-Aug. 1945<br>May 1941-Dec. 1941 |
| Leyte-Samar | 215,603,201 | Oct. 1944-Aug. 1945 |
| Manus | 131,757,843 | March 1944-Aug. 1945 |
| Okinowa | 113,715,453 | April 1945-Aug. 1945 |
| Saipan | 63,252,622 | June 1944-Aug. 1945 |
| Trinidad | 45,704,394 | April 1941-May 1944 |
| Argentia | 44,704,394 | Dec. 1940-Sept. 1944 |
| Espiritu Santo | 36,369,925 | June 1942-June 1946 |
| Tinian | 35,158,275 | July 1944-Aug. 1945 |
| Bermuda | 34,983,064 | Feb. 1941-Jan. 1945 |
| Subic Bay | 34,916,587 | Jan. 1945-Aug. 1945 |
| Noumea | 24,297,449 | Nov. 1942-Jan. 1945 |
| Eniwetok | 23,085,079 | Feb. 1944-Aug. 1945 |
| Ulithi | 18,580,273 | Sept. 1944-Aug. 1945 |
| Peleliu | 18,489,159 | Sept. 1944-June 1945 |
| Iwo Jima | 15,462,733 | Feb. 1945-Aug. 1945 |
| Manila | 13,349,589 | March 1945-June 1945 |
| Milne Bay | 11,101,000 | May 1943-July 1945 |

* Reference IX-10 (Building the Navy's Bases in World War II 1947)

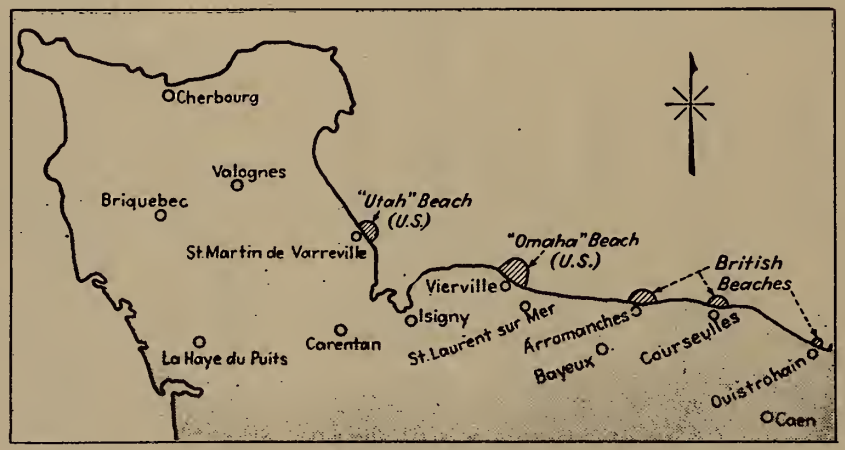



FIG. IXB-1 - The location of the invasion harbors in the Bay of the Seine during the Normandy landings. The main artificial harbors were at St. Laurent sur Mer and Arromanches. (72)




FIG. IXB-2 - Midway Islands (33)

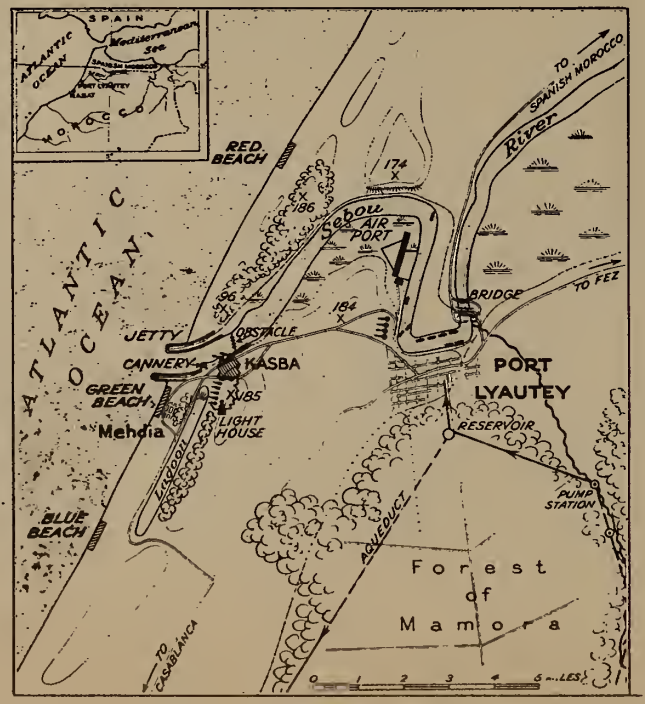



FIG. IXB-3 - Port Lyautey, Morocco, and vicinity (37)



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# MANUAL OF AMPHIBIOUS OCEANOGRAPHY

## SECTION IX: INVASION HARBORS

## C. GENERAL FEATURES OF A HARBOR

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## C. GENERAL FEATURES OF A HARBOR:

All harbors have some type of structures (such as piers, breakwaters, jetties, seawalls and groins) to provide rapid and safe ship-unloading and to protect the harbor and shoreline. These structures are discussed in Sections E through I. Other important harbor features to be considered are the entrance channel width, the effect of waves inside a harbor, and the foundation for the structures. Harbor surge should be avoided if possible. The possibility should not be overlooked of using models of harbors and breakwaters to solve some of these more general problems -- especially when the overall effect of waves on the harbor, the structures, and the coastline is not readily determined by observations and calculations.

### 1. Width of Channel:

The calculation of the width of the entrance channel is independent of the type of breakwater and entrance construction. The important consideration is the width necessary for a safe shipping entrance balanced by the width necessary to cause scour of channel to the desired depth. (Dredging is expensive, slow, and somewhat impractical.) This latter consideration depends on the tidal prism. The larger the tidal prism, the higher the channel velocity for a given channel width. When inner harbor improvements decrease the tidal prism, the channel velocity becomes less and it may become necessary to increase dredging operations to keep the channel clear. The size of the channel can be extimated from the following empirical equation:

$$A \cong 1000\, V^{0.85} \qquad \text{IX C-1}$$

Where V = Volume of the tidal prism between MLLW and MHHW in the square mile-feet.

A = area of the entrance channel section below mid-tide, in square feet.

It should be remembered that indiscriminate dredging of channels and harbors may cause undesirable saline contamination of local ground water.

### 2. Effect of Waves in a Harbor:

There are several common causes of waves in a harbor:

(i) wind

(ii) wave passage through the harbor entrance, and

(iii) wakes of ships

The effect of wind waves can be determined by wave forecasting (See Manual, Section III). The reduction in wave height of waves passing through the harbor entrance can be calculated by using refraction and diffraction diagrams. (See Manual Section ID and IE). The waves in the wake of ships depend upon the size of the ship and its speed, and the solution of such problems may be found in naval architecture literature.

3. Harbor Bottoms:

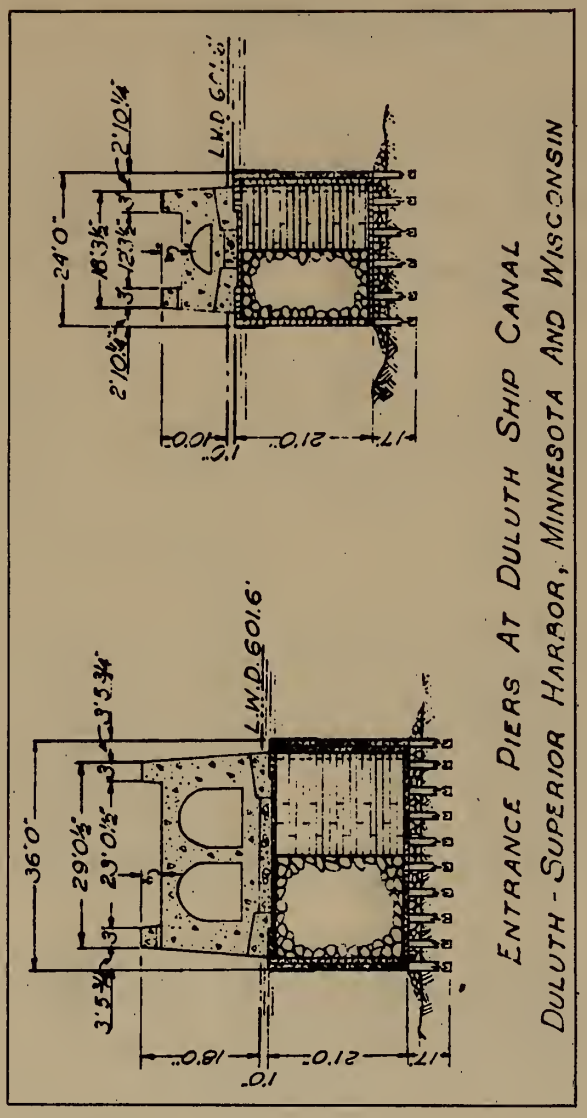
The harbor bottom must provide good anchorage for ships, and a good foundation for the protective structures.

(i) If the bottom is rock or firm hard soil and rock, then a rubble-mound breakwater, wall-type breakwater or similar structure can be laid directly on the bottom, providing the bottom is made nearly level. With this type of bottom, driving pile is difficult or impossible.

(ii) If the bottom is moderately firm, a foundation of rubble will be needed to support wall-type breakwaters. Sometimes a blanket of sand is laid and then the rubble.

(iii) If the bottom is soft and muddy - but the mud is not too deep, a trench is dug and rubble filled in to support wall-type breakwaters. For deep muddy bottoms, it is necessary to drive piling (wood or concrete) to create a foundation for a wall-type breakwater, and to help support a rubble mound. (Examples of piling supports for breakwaters are shown in Figure IX C-1).

(iv) If the bottom is sand that needs to be confined, it may be necessary to drive sheet piling. A fascine is sometimes placed on sand (made up of several layers of logs lashed together - with branches interwoven). The fascine is floated into position, sunk with rocks, and allowed to fill up with sand before a rubble mound is laid.



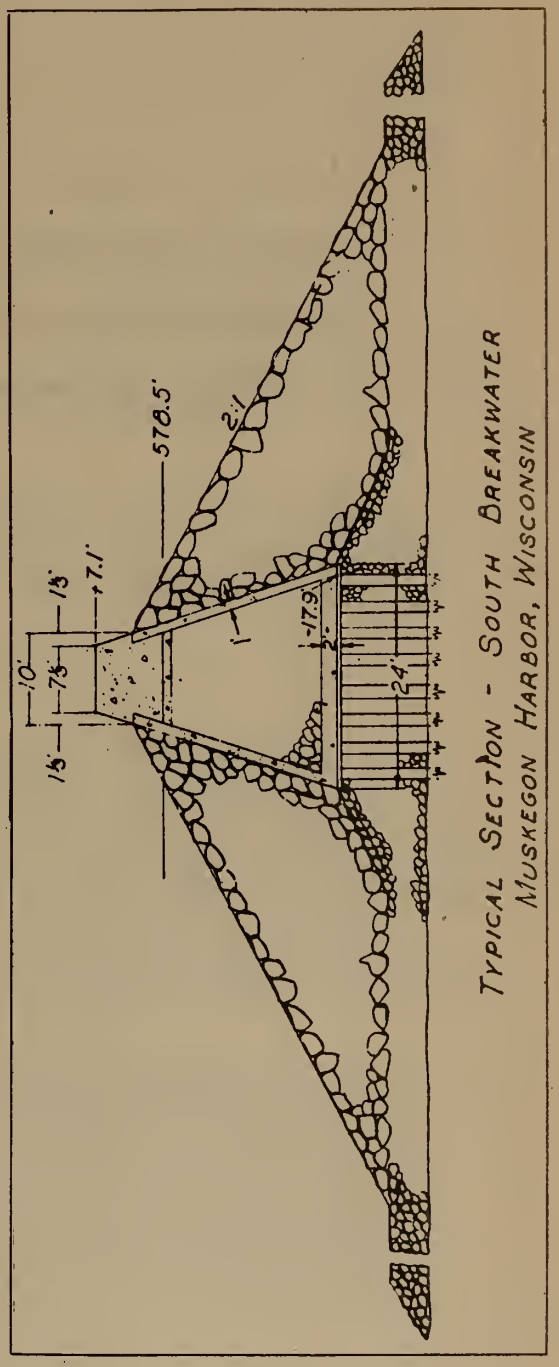




FIG. IXC-1 - Pile supports for breakwater formation (23)

# MANUAL OF AMPHIBIOUS OCEANOGRAPHY

## SECTION IX: INVASION HARBORS

## D. THE EFFECT OF WAVES

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## D. THE EFFECT OF WAVES

The United States has carried out more successful invasions than any other country, but even with this vast experience, the success of an invasion still depends on correct prediction of two of nature's most destructive forces--storms and waves. Whether a permanent harbor structure can withstand wave action depends, not only on the method of construction and design, but also on the correct determination of the expected wave action.

Since the purpose of building breakwaters, jetties, seawalls, and groins is to protect harbors and shorelines from waves and currents, then the need for a particular structure at a given location should be determined by the severity of the waves or currents and the frequency of the severest conditions. Before deciding to build any such structures, a thorough study of the wave history should be made. This prior study is particularly important when one considers the high cost and long time involved in building these structures.

For example it took several years and cost many millions to build the rubble breakwater at Los Angeles Harbor. The temporary harbors for the invasion of Normandy were planned for several years, and mostly built in eight months, erected in about one month, and maintained and expanded for four months at a cost of hundreds of millions of dollars.

### 1. Nature of Wave Study:

For any given location, the wave conditions can be determined by hindcasting from historical weather maps (if available). The wave climate in some locations has been and is being observed and these historical records will aid in the wave determination study. Information as to available wave hindcasts (and forecasts) and historical records may be obtained from the U. S. Hydrographic Office, the Corps of Engineers, the U. S. Weather Bureau, and other related offices. If no information is available, then one can make an approximate computation of the maximum possible wave conditions from the hydrography and forecasting theory (see Section III of Manual on Forecasting).

A wave study should include the effect of the hydrography on wave refraction and diffraction by structures inside and outside the harbor. The refraction and diffraction of waves are described in another section of this manual. Points of wave concentration should be investigated and avoided, if possible, as sites of construction. If the harbor is to be built to supply forces for an invasion, a study of wave and weather offshore is necessary so that towing of prefabricated parts of the harbor can be made during periods that are relatively safe from severe storm conditions.

Table IX D-1 shows limitations that wave action put on ocean operation. Invasion operations are difficult if waves become 3 to 4 feet high or excite the natural frequencies of the ship. If waves became 6 feet high, practically all operations would cease. Figure IX D-1 shows the relationship of surf to discharge of cargo during the month of October, 1944 for part of the Normandy Invasion. When local storms occur and persist and the waves become 8 feet or more in height, then temporary and poorly protected permanent harbors will probably be damaged or possibly destroyed.

2. Example of Daily Wave History:

As an example of what can be done and was done during the invasion of northern France, the daily wave history was recorded and summarized by the late H. R. Seiwell (Reference IX-73):

"A review of oceanographic conditions on the target beaches during June 6 and 7 show that sea conditions were none too favorable for landing men and equipment. In the early hours of June 6, a gusty northwest wind (12 to 18 knots) produced wind waves averaging 3 to 4 feet, with occasional maxima of 6 feet in the embarking area some 10 miles offshore. The short periods of the waves made transfer of men and equipment into landing craft difficult. On Omaha Beach, breaker heights of 3 to 4 feet, and on Utah Beach, breaker heights of 2 feet were recorded for the initial landings; by 1500 hours, the surf on Omaha Beach appeared to have dropped to less than 3 feet. In the channel, wind waves of 3 to 4 feet continued throughout the morning of June 7, and by mid-afternoon the northwest wind dropped to 5 to 10 knots and the wind waves to 2 to 3 feet, by 1800 hours on June 7, the surf on Omaha Beach had dropped to 1 to 2 feet ...

It is of interest to note that surf on the beaches and wind waves in the open channel were forecasted with a reasonable degree of accuracy. The Hydrographic Bulletin for June 6, disseminated previous to midnight of June 5, forecasted an average surf height on the beaches of 3 feet with a maximum of 4 feet and average wave heights of 3 to 4 feet in the open channel ...

For the ten-day period following D-Day, state of the sea and surf was of no particular hindrance to maintaining a continuous stream of supplies from ship to shore. The maximum wind wave height in the channel was 2 to 3 feet, and surf on the beaches was slight, less than 2 feet.

On June 17 the wind shifted into the north-north-east and attained velocities up to 18 knots, sufficient to cause 3 to 4 foot wind waves in the exposed channel and a 3 foot surf on the beaches. The following day, wind and sea dropped, although a maximum of 3 foot surf on the beach was continued. Unloading of cargo was impeded on the 19th when increased northerly winds produced moderate to rough seas of 4 to 6 feet in the exposed unloading zone, and a maximum 6 foot surf on the beaches. During the next two days, sea conditions became increasingly worse; wind gusts were in excess of 20 knots and wind waves in the unloading zone were 6 to 7 feet, while surf on the beaches was observed to be 5 to 6 feet inside the sector protected by the block ships and 8 to 9 feet along unprotected portions of the beaches. During this two-day period of intense surf, confused breakers resulting from the uneven contours of the beaches and the rapidly changing tides, increased its effective intensity; cargo operations across the beaches were practically at a standstill. The following day (June 22), the sea abated and by midafternoon wind waves in the channel were a moderate 5 feet with breakers inside the protected mulberry sector, a maximum

of 3 feet and 5 feet along exposed portions. By June 23 wind wave in the channel and surf on the beaches had decreased to 1 to 2 feet ...

Thus wave heights as occurred during June 1944 in the English Channel were not unique and had been recorded for this same season of other years." "...Wave heights in excess of 8 feet had an occurrence expectancy of approximately 5 percent during the month of June."

3. Examples of Wave Destructiveness:

The force of waves is demonstrated by failures of past constructions caused by waves, and also by the destruction by waves of natural landmarks.

In 1872, at Wick Bay in Scotland, waves moved a 1,350-ton section of the Wick Breakwater. A cross-section of this breakwater is shown in Figure IX D-2.

In February 1934, a wall-type breakwater completely failed in Algiers. This was the Mustapha breakwater - designed for 16 foot waves - which was subjected to a storm with waves 21 feet high during constuction, and finally destroyed by a severe storm where the maximum waves were said to be 30 feet high.

"After the crest of an enormous wave passed over the breakwater without breaking and before the passage of the following trough, the superstructure was clearly seen to have resisted the force of the wave. Then a slight trembling of the superstructure was noted, followed by the complete collapse of the breakwater as the trough of the wave passed".

(Quoted from Reference IX-49, Kaplan and Pope, 1950)

On the North Pacific coast, at the mouth of the Columbia River, 20 feet waves are common occurrences. This type of furious wave action on the end of the south jetty of the Columbia River, unravelled the jetty and it was necessary to build a tremendous solid concrete terminus to stop the unravelling. This terminus is shown in Figure IX D-3.

At Buffalo, New York in 1899 and 1900, falling water from wave action on breakwaters damaged and broke timber that was 12" x 12" x 12". Figure IX D-4 shows some of the damage at Buffalo.

A 20 ton block was lifted by a wave to a height of 12 feet and landed upon the top of a pier which was 4 feet 10 inches above high water at the nearly vertical breaker of Ymiridan, the harbor entrance of the Amsterdam Canal, Holland.

At Tillamook Rock, Oregon, December 1894, waves threw a 135 pound rock so that it fell through the roof of the lightkeeper's house which was 91 feet above low water. The lantern of the light house was broken at an elevation of 139 feet above low water (7 foot tides).

The Standard Oil Company pier at El Segundo, California is opposite a submerged ridge at about the 12 fathom depth. Long period waves (18 seconds) from the Northeast converge near the pier. Waves 20 feet high have been observed at the pier when a short distance away on both sides, the waves were not more than 6 feet high. High waves have destroyed the outer end of the pier.

Probably the most recent unique failure of an entire harbor was the Normandy Invasion harbor at St. Laurent sur Mer and the partial damage at Arromanches. These harbors were designed for construction and operation during 3 and 4 foot waves of 5 to 6 second period and to withstand waves of 8 to 10 feet in height with the same period. The harbors were partly constructed and operating according to plan for about 16 days which was an engineering feat heretofore unequalled. Then a storm as described previously struck the harbor. (Further description of the failure is given in the section on breakwaters.)

Another rather unusual type of wave damage (unusual because the structures are somewhat unique) was that caused by 27 to 35 foot waves crashing against the oil drilling platform in the Gulf of Mexico during the hurricane of October 3 and 4, 1949. Figure IX D-5 shows one structure before the storm and Figures IX D-6 and 7 show some of the $200,000 worth of damage after the storm. Figure IX D-7 shows all that was left of the platform and buildings after the storm (that were shown before the storm in the upper left portion of Figure IX D-6). Figure IX D-8 shows the damage to another platform where the part of the pile structure broke loose at the bottom of the sea. Notice the close spacing of the piling on the structures in Figure IX D-8.

A recent breakwater failure due to large storm waves and storms of considerable duration is shown in Figure IX D-9 of the end of the Crescent City breakwater. The type of wave damage that occurs to seawalls is shown in Figures IX D-9 to 13. The type of damage to a groin is shown in Figure IX D-14.

TABLE IX D-I

Summary of Generalized Performance Data for Marine Operations
(From Reference IX-25, Fleming and Bates, 1950)

| Type of Operation | Wave Height (in feet) for: Safe, Efficient Operation | Marginal Operation | Dangerous and/or Inefficient Operation |
|---|---|---|---|
| Deep Sea Tugs: | | | |
| Handling oil and water barge | 0-2 | 2-4 | 4 (or more) |
| Towing oil and water barge | 0-4 | 4-6 | 6 " |
| Handling derrick barge | 0-2 | 2-3 | 3 " |
| Handling and towing LST-type vessel | 0-3 | 3-5 | 5 " |
| Crew Boats 60-90 feet in length | | | |
| Underway | 0-8 | 8-15 | 15 " |
| Loading or unloading crews at platform | 0-3 | 3-5 | 5 " |
| Supervisor's Boats -- fast craft 30-50 feet in length | | | |
| Underway at cruising speed | 0-2 | 2-4 | 4 " |
| Loading or unloading personnel at platform or floating equipment: | 0-2 | 2-4 | 4 " |
| LCT-type Vessel and Cargo Luggers | | | |
| Underway | 0-4 | 4-5 | 5 " |
| Loading or unloading at platform | 0-3 | 3-4 | 4 " |
| Loading or unloading at floating equipment | 0-4 | 4-5 | 5 " |
| Buoy Laying (using small Derrick Barge) | 0-2 | 2-3 | 3 " |
| Platform Building | | | |
| Using ship-mounted derrick | 0-4 | 4-6 | 6 " |
| Using large derrick barge | 0-3 | 3-5 | 5 " |
| Pipe-line construction | 0-3 | 3-4 | 4 " |
| Gravity-meter exploration using surface vessel (limiting conditions caused by instrument becoming noisy) | 0-4 | 4-6 | 6 " |
| Seismograph Exploration using craft under 100 feet in length | 0-6 | 6-8 | 8 " |
| Large Amphibious Aircraft (PBY) | | | |
| Sea Landings and Take-offs | 0-1.5 | 1.5-3 | 3 " |
| Boat-to-plane transfer operations in water | 0-1 | 1-2 | 2 " |
| Small Amphibious Aircraft | 0-1 | 1-2 | 2 " |

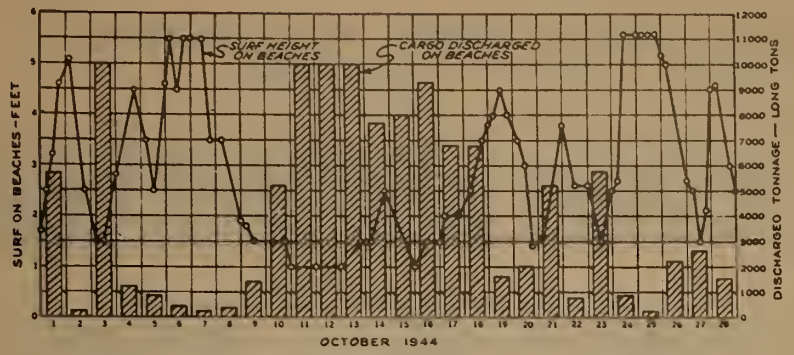



FIG. IX D-1 - Observed relationships between surf heights and daily discharge tonnage, Ohmaha Beach, 1-28 Oct 1944 (24)

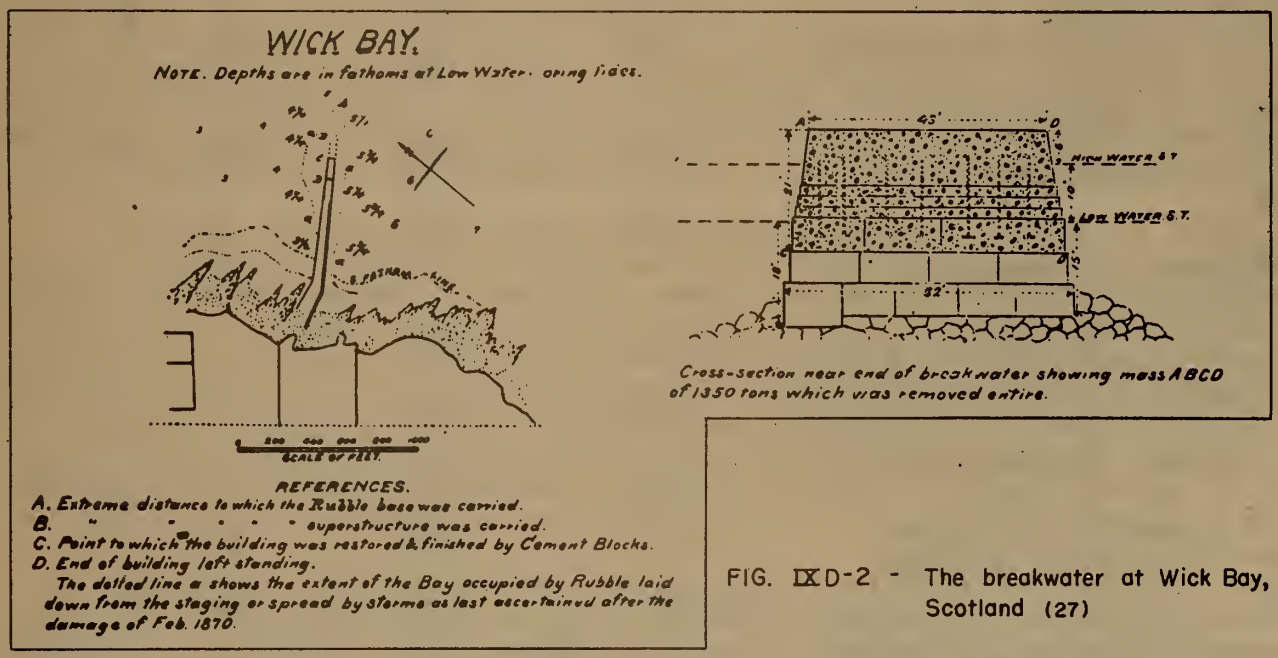



FIG. IX D-2 - The breakwater at Wick Bay, Scotland (27)

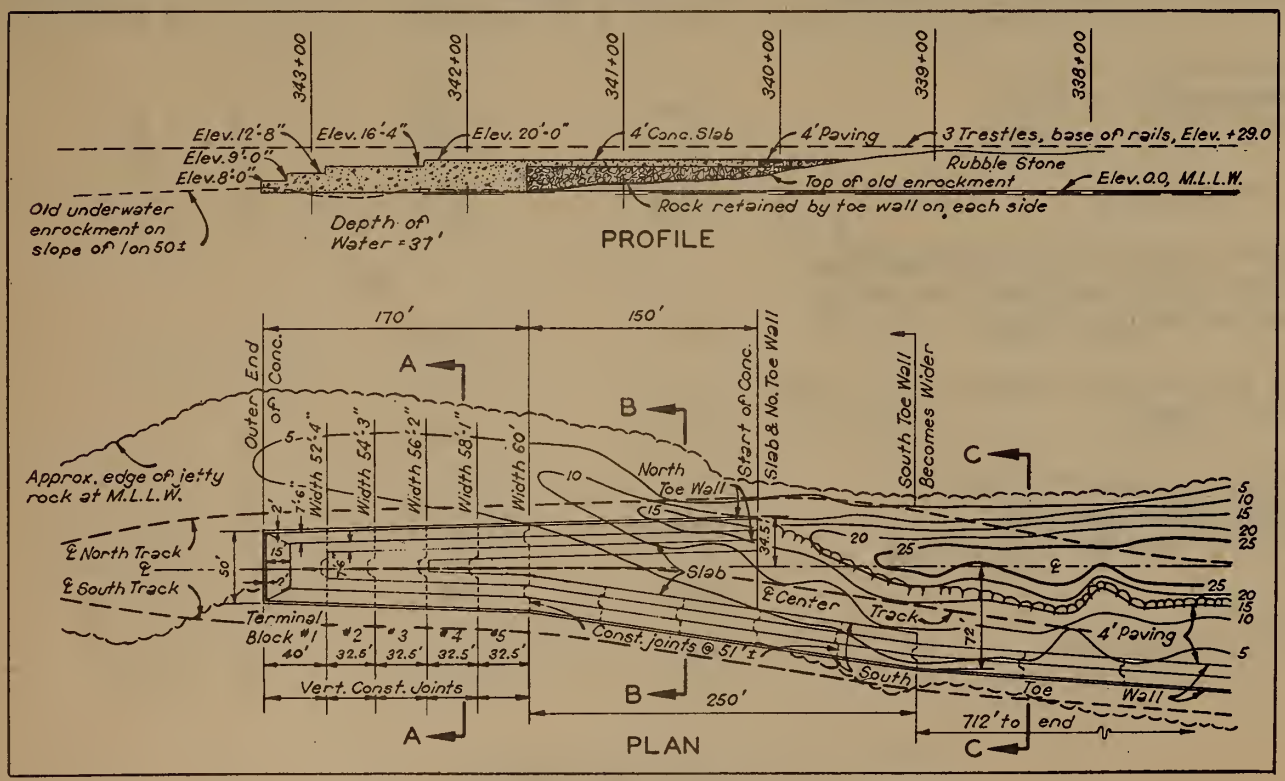



FIG. IX D-3 - South jetty terminal, mouth of Columbia River, Oregon and Washington (39)



FIG. IX D-4 - Breakwater damaged by water falling from waves, Buffalo, N.Y. (27)



FIG. IX D-5 - Ohio Oil Co. drilling structure before the hurricane of 3 October 1949 (24)

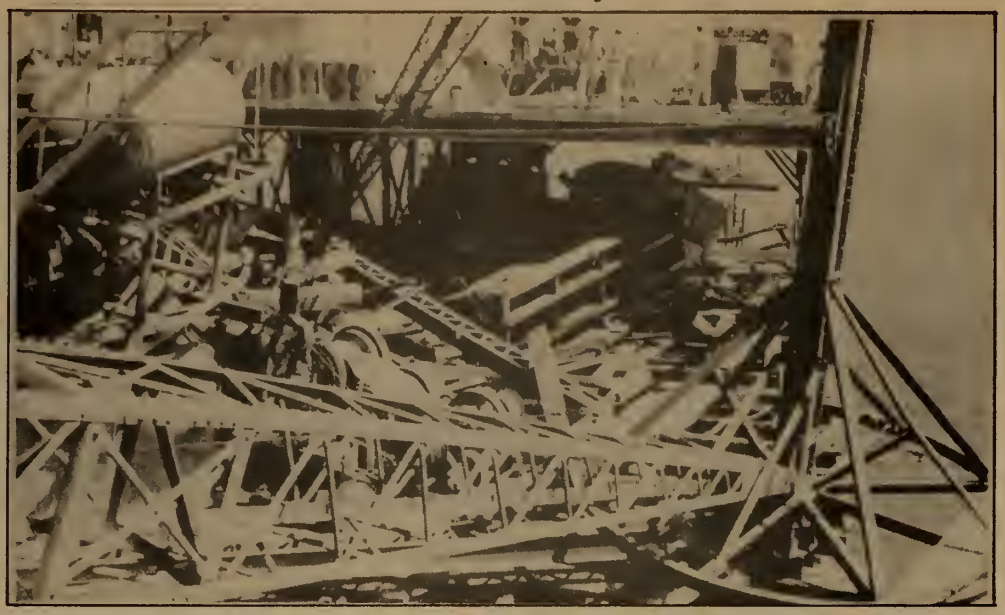


FIG. IXD-6 - Ohio Oil Co. main drilling platform after hurricane (69)



FIG. IXD-7 - Platform after hurricane (12)

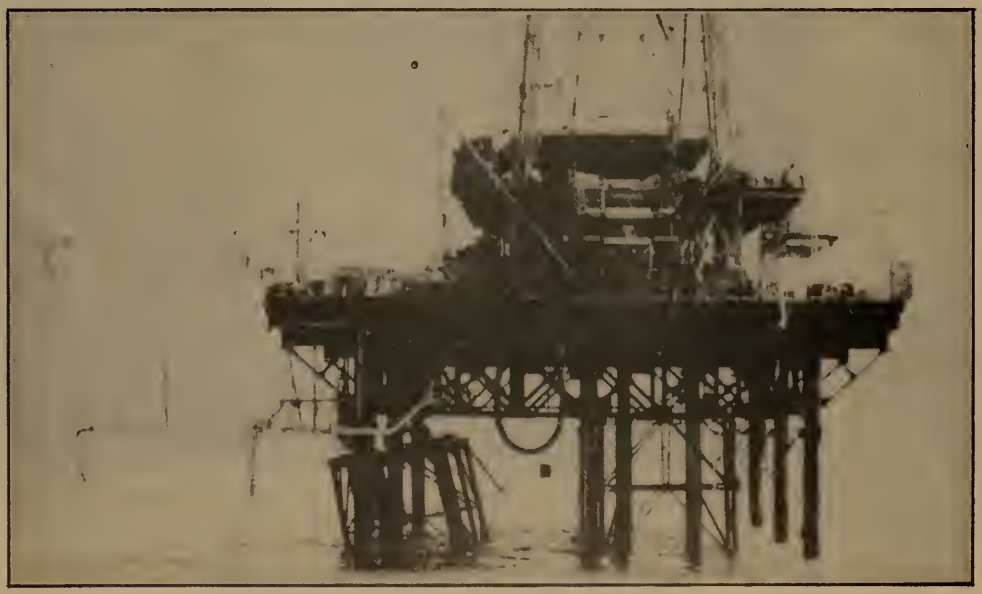


FIG. IXD-8 - Stanolind Oil & Gas Co. drilling platform after hurricane (2)



FIG. IX D-9 - Damage to the Cresent City breakwater (17)



FIG. IX D-10 - Wave-damage to bulkhead (22)



FIG. IX D-12 - Damage caused by waves over-topping wall at Point Beach, Milford, Connecticut (22)



FIG. IX D-14 - High groin with ruined low groin in foreground, Seabright, N. J. (22)



FIG. IX D-11 - Battered bulkhead (22)



FIG. IX D-13 - Damage to curved-face concrete wall (22)

# MANUAL OF AMPHIBIOUS OCEANOGRAPHY

## SECTION IX: INVASION HARBORS

### E: PIERS

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## E. PIERS

The chief purpose of a harbor or of harbor protection is to permit the loading and unloading of ships. Piers must be built to make these operations safe and rapid.

During the first phase of an assault landing, no piers are available immediately. An LST may beach and have a bulldozer (previously discharged from an LST) push out a dirt and rock pier if the soil will permit such an operation. A beached LST may discharge a pontoon causeway or may beach at a previously constructed pontoon causeway. Several varieties of assault piers have been shown previously (Figures IX A-7 to 19).

In the second phase of an assault (primarily one of supply) piers should be improved, although in the past this has not always been done. Piers should be built as rapidly and durably as possible. Whenever possible, they should be protected by temporary breakwaters. Figure IX A-13 is an example of an unprotected pier.

Protected pier sites were planned and constructed during the Normandy invasion. Figure IX E-1 shows the harbor at Omaha Beach opposite St. Laurent sur Mer. Figure IX E-2 shows Utah Beach. Figures IX A-1 and E-3 show the greatest of all temporary harbors at Arromanches.

Probably as outstanding as the artificial harbors used in the Normandy invasion were the semi-floating pierheads (Figures IX E-4, 5 and 6) and the floating roadway (Figure IX E-7). Figure IX E-8 shows the pierheads and floating road during a test. The parts were made, floated, towed to the site of the invasion and installed. Figure IX E-9 shows a model towing test of the floating road. Figures IX E-10 to 14 show detailed construction of the floating roads (Whales).

Figures IX E-15 and 16 show details of the construction of the type of pierheads where floating or non-floating causeways may be used.

In addition to the temporary piers of the second phase of an invasion, there are the permanent type of pier of the third phase. The prefabricated Baily bridge units are used in an unique fashion to construct a pier as shown in Figure IX E-17. Conventional type piers of permanent harbors must be repaired or built. Since construction is so varied, no specific details can be given except that pile driving and dredging will probably be needed.

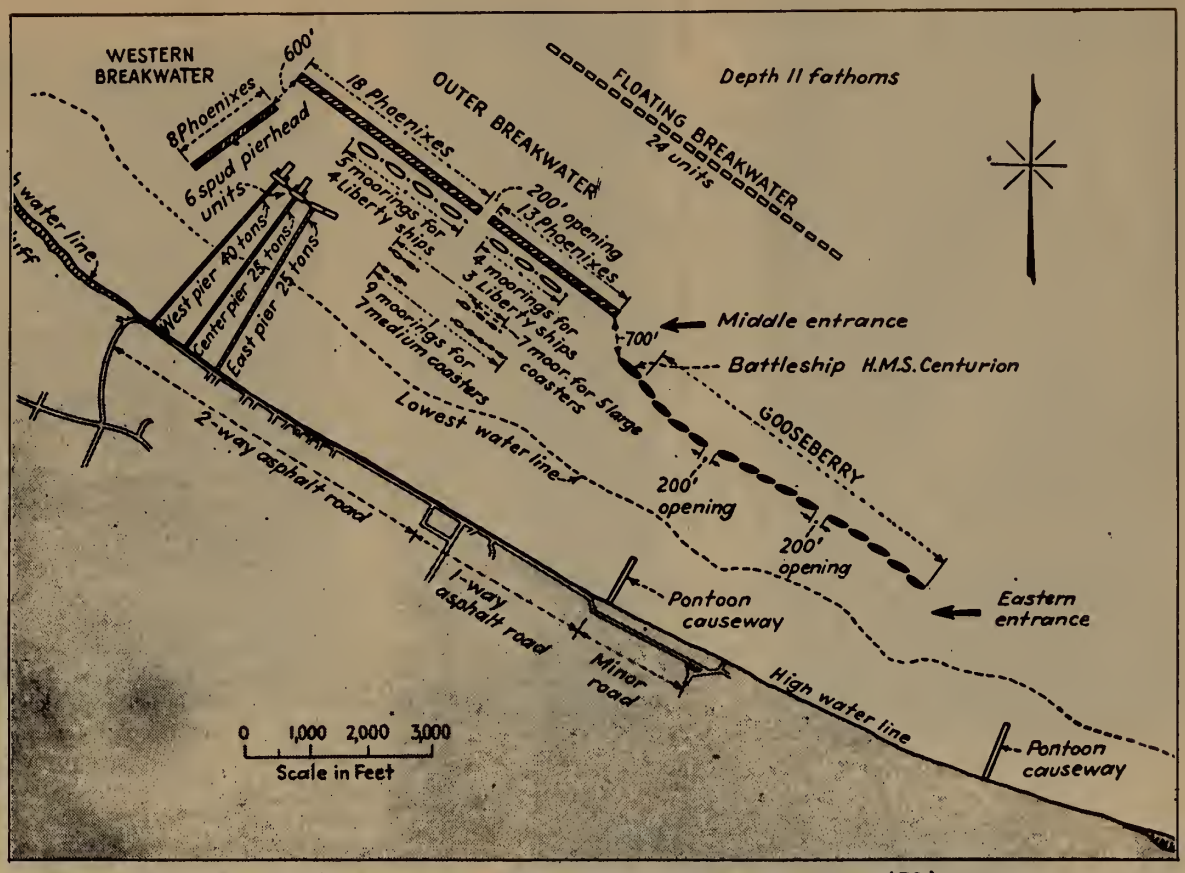



FIG. IX E-1 - "Omaha" beach, Mulberry A, at St. Laurent sur Mer (72)

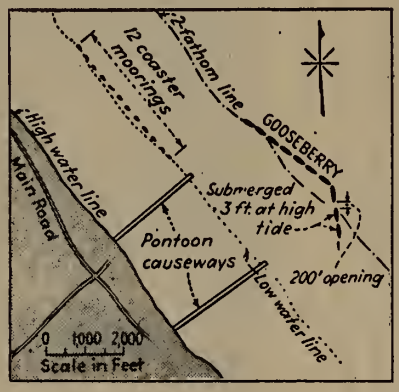



FIG. IX E-2 - "Utah" beach sunken ship breakwater (72)

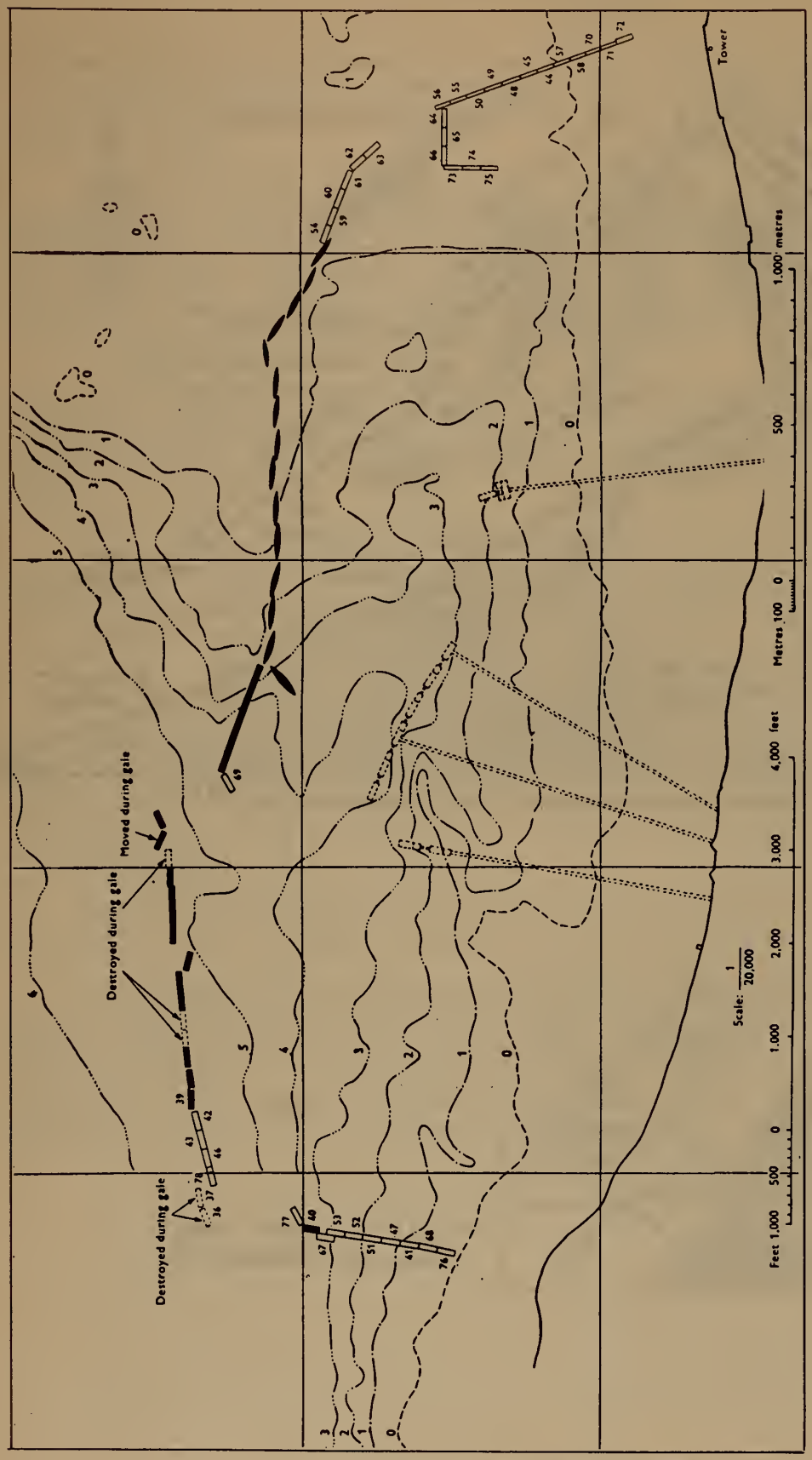



FIG. IXE-3 - Harbor at Arromanches, Mulberry B (46)

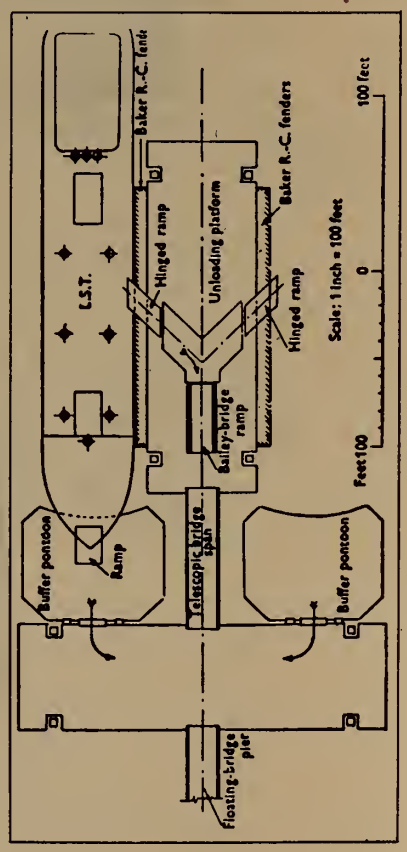



FIG. IXE-4 - LST pierhead (67)



FIG. IXE-5 - LST pierhead showing Baker fenders and platform, hinged ramp raised (67)



FIG. IXE-8 - Floating roadways in use at Arromanches (90)

FIG. IXE-9 - Model towing test of floating roadway (80)---



FIG. IXE-6 - Intermediate pontoon positioned in pierhead assembly (90)



FIG. IXE-7 - Sea test under storm conditions of floating roadways and pierhead (6)





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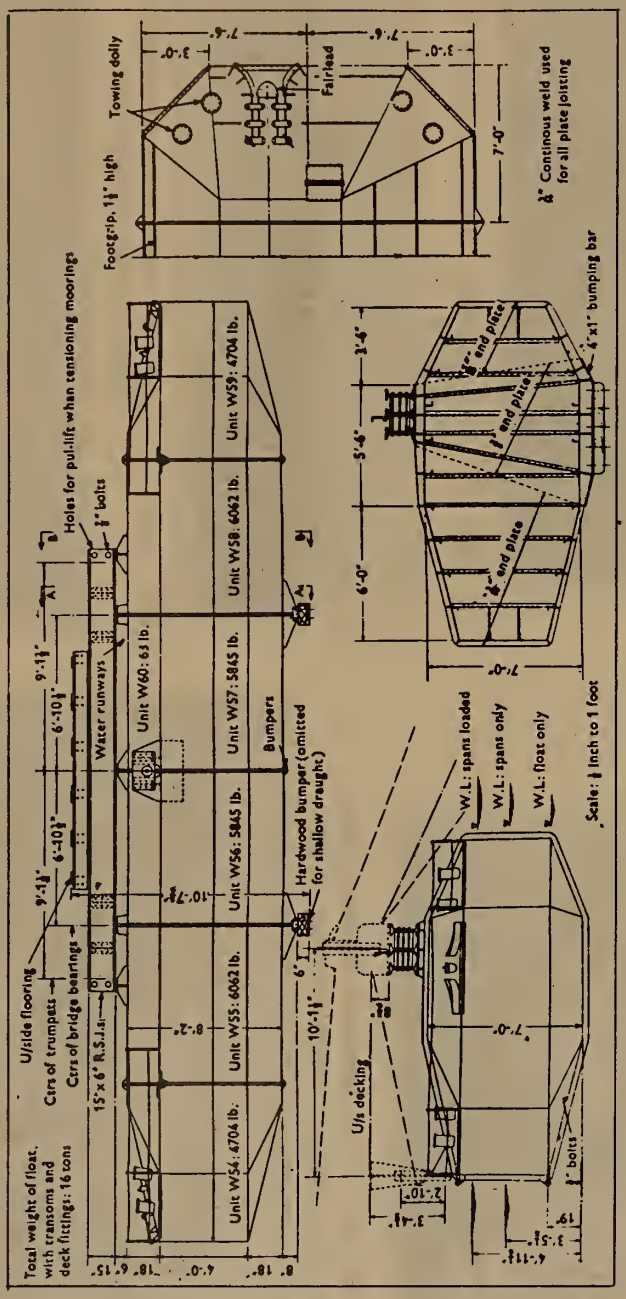



FIG. IXE-10 - Steel pontoon for Whale floating bridge (6)

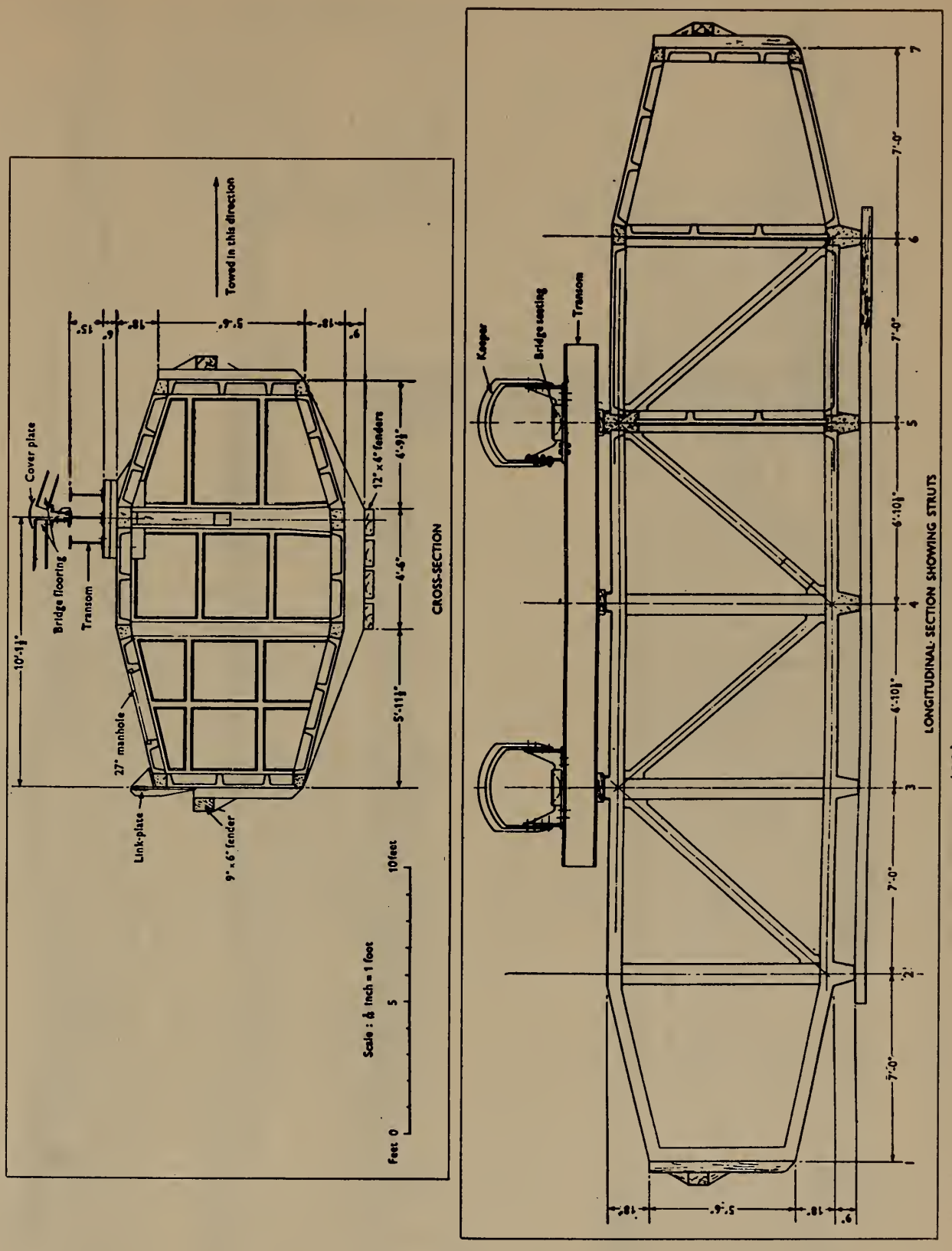



FIG. IXE-11 - Reinforced concrete pontoons (90)

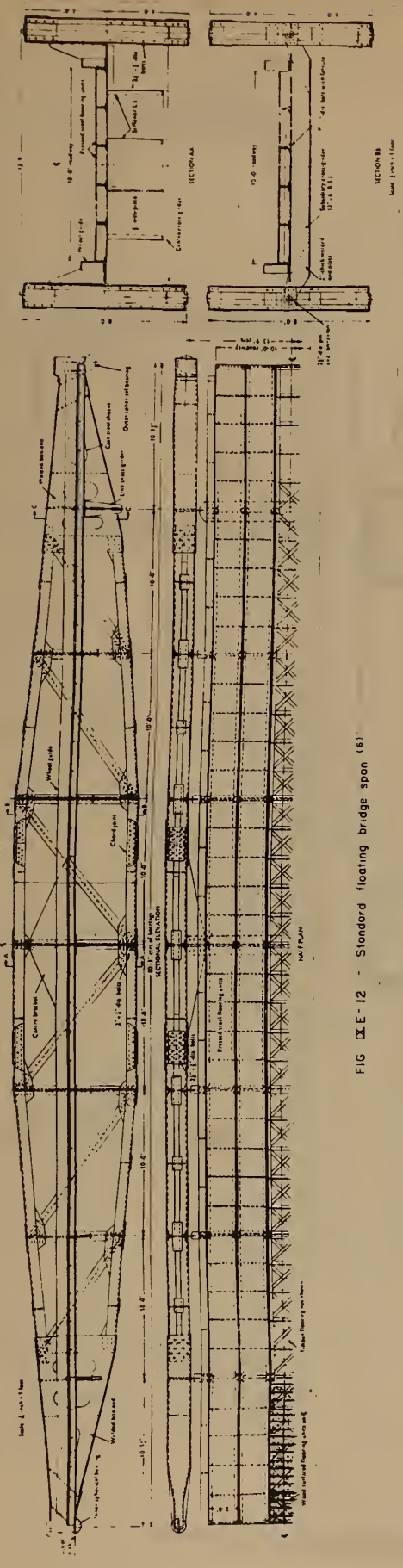


FIG IX E-12 - Standard floating bridge span (6)

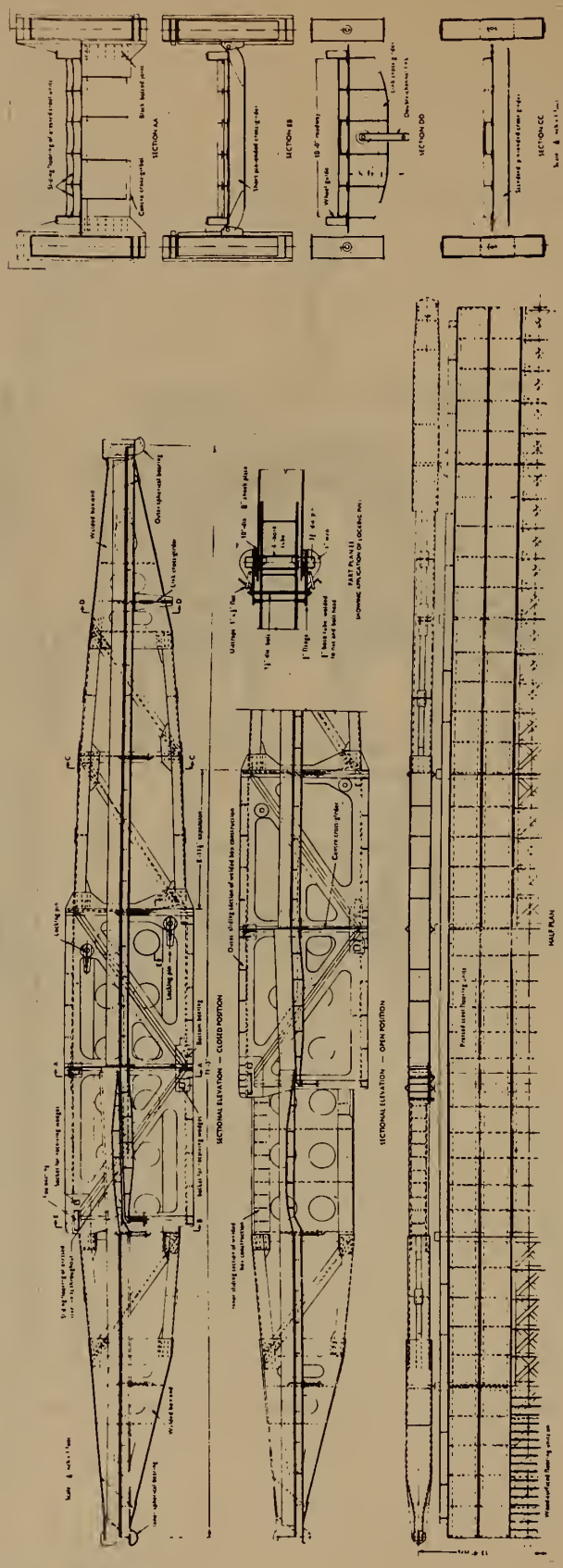


FIG IX E-13 - Telescopic span for deep water floating bridge (6)




FIG. IX E - 14 - Bearings allowed for waves with L/h = 9 (24° between lines of the flooring) (6)

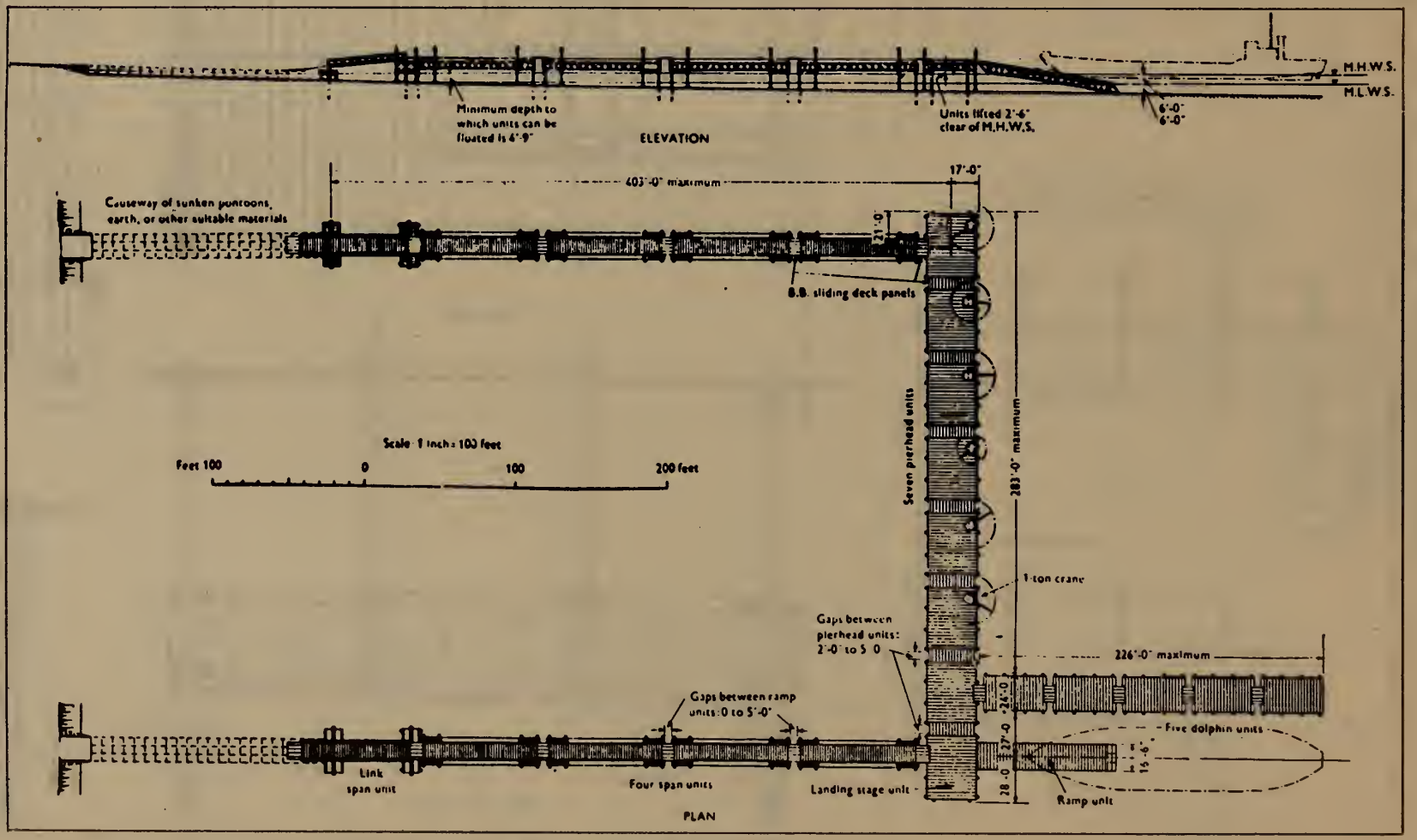



FIG. IX E - 15 - Standard lay-out of a composite pier (66)



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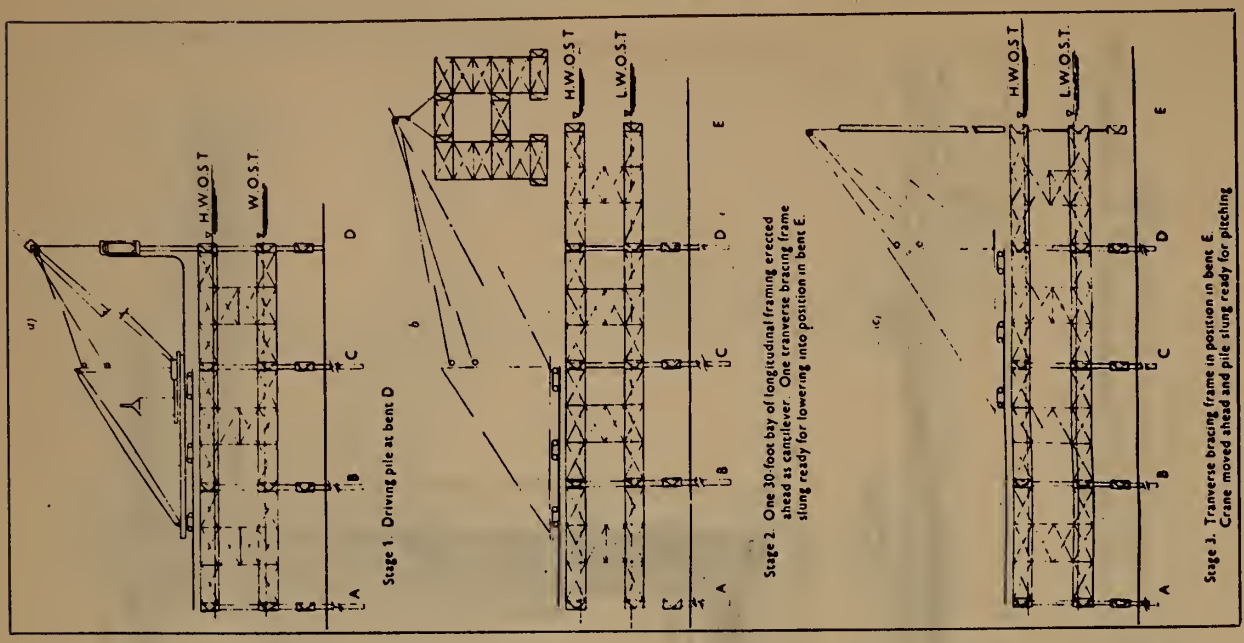



FIG. IXE-17 - Baily bridge-pier erection (84)

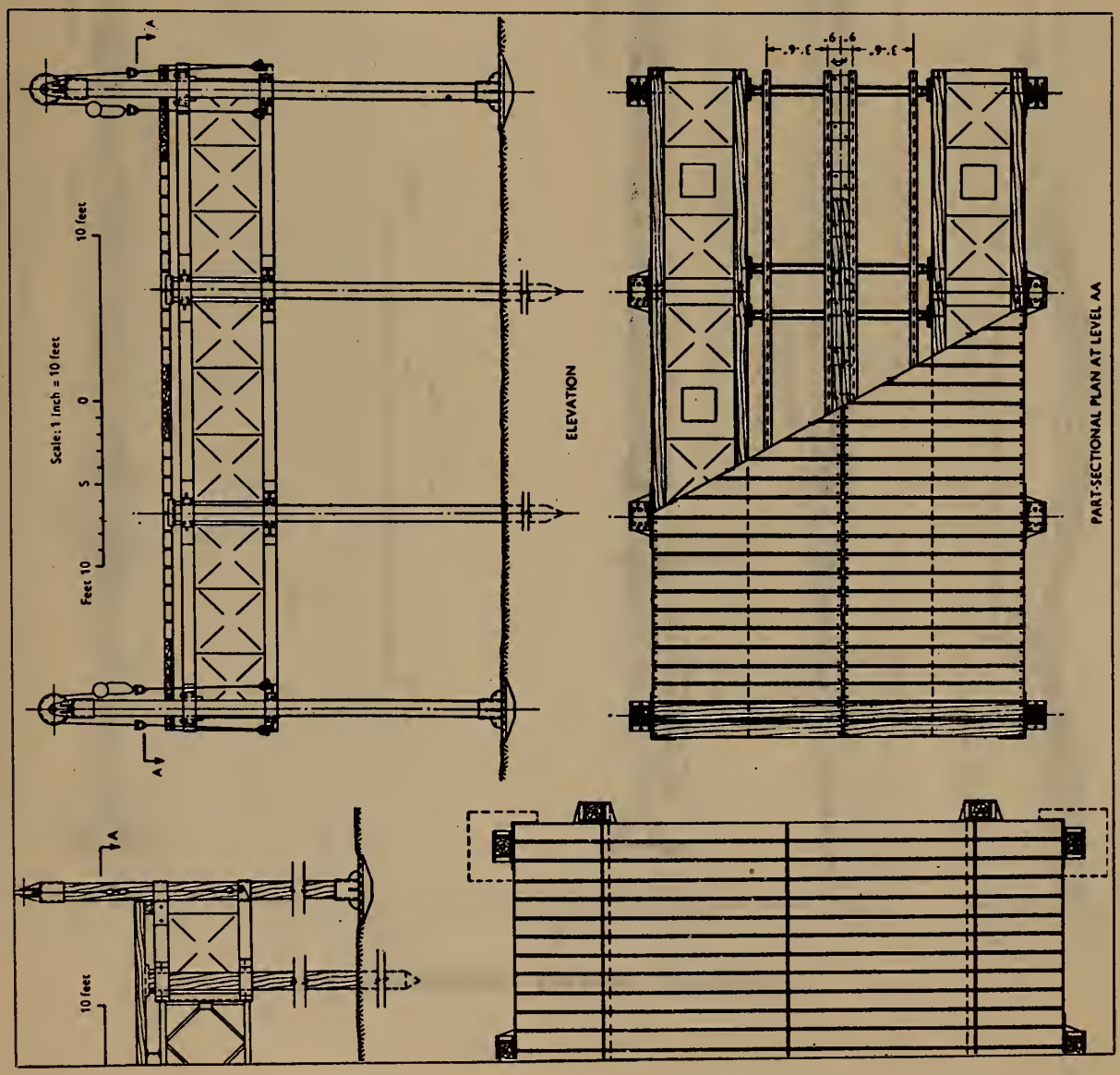



FIG. IXE-16 - Composite pierhead (66)

# MANUAL OF AMPHIBIOUS OCEANOGRAPHY

## SECTION IX: INVASION HARBORS

## F. BREAKWATERS

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## F. BREAKWATERS

The purpose of a breakwater is to protect a harbor from wave action by reflecting or dissipating some or all of the wave energy. There are several types of breakwaters, and specific reasons for the use of each type. Choice of a breakwater (its type, size, and method of construction) depends on the following factors:

- the nature of the harbor: its site and size; whether it is military or civilian, temporary or permanent;
- the nature of wave action, and the direction of the severest storms; the hydrography of the site; and
- the availability of building equipment, and the expected maintenance cost (Permanent breakwaters are built with a calculated amount of yearly repair, plus some calculated amount for major repair - - say 25% every 15 years).

The temporary invasion harbor is built to protect ships and piers during the second phase of supply for an invasion force. (However, in many instances, the second phase consists of improving piers and pier sites without supplying any artificial protection such as breakwaters.) Each site of an invasion harbor presents newproblems, but to date the breakwaters used have been sunken caissons, sunken ships, and floating breakwaters. Several other types have been considered but never used -- including underwater breakwaters and air breakwaters.

The purpose of this summary is to indicate the various types of temporary harbor constuctions, including several types that have not been used, and to describe the various types of permanent harbor constructions.

1. Sunken Caissons:

The sunken caisson breakwater consists of individual caissons anchored close together and in line with previcusly sunken caissons. Figure IX F-1 shows the caissons in place at Arromanches during the invasion of Normandy, notice the effectiveness of this temporary breakwater. The caissons are constructed of concrete and towed to the site. Figure IX F-2 shows the caissons of one Normandy invasion in a storage zone before being towed to the harbor site.

The caisson is sunk by opening valves and allowing sea water to fill the compartments. Figure IX F-3 shows the positioning and sinking of a caisson. The water remains inside the caisson, giving it enough added mass so that it is not essential to fill the caissons with rock, sand, or concrete. However, such filling is desirable for added stability.

In building caissons, the following factors should be considered:
(i) The depths of water in which they are to placed. A caisson must extend about one half the expected wave height above the highest tide stage.

(ii) The amount of over-turning caused by wave action.

(iii) The anchors necessary to resist the thrust caused by wave action, and

(iv) The type of foundation upon which the caissons rest -- with particular reference to over-turning and settling of the caissons caused by the rocking of the structures by the waves and by scour.

The type of caisson used in the Normandy Invasion was tested several times and shown to be adequate for the designed wave action. Figures IX F-4, 5, 6, 7, and 8 show the detail of the various sizes of caissons used.

Table IX F-I shows the sizes of the walls and floors of the caissons.

TABLE IX F-I
(Reference IX-89 Wood. 1948a)

| Unit. | Floor: inches. | External Walls. | | Internal Walls. | | Floor of swim end: inches. |
|---|---|---|---|---|---|---|
| | | Lower: inches. | Upper: inches. | Longitudinal: inches. | Cross: inches. | |
| A1 | 15 | 15 | 14 | 9 | 9 | 12 |
| A2 | 15 | 15 | 14 | 9 | 9 | 12 |
| B1 | 12 | 13½ | 13½ | 9 | 9 | 11 and 10 |
| B2 | 12 | 12 | 12 | 9 | 9 | 11 and 10 |
| C1 | 12 | 13½ | -- | 9 | 9 | 9 |
| D1 | 12 | 12 | -- | 9 | 9 | 12 |
| Ax | 15 | 15 | 14 | 9 | 12 and 9 | -- |
| Bx | 12 | 13½ | 13½ | 9 | 9 | 11 and 10 |

Figure IX F-9 shows caissons under construction before launching. The test set-up for the caisson model studies are shown in Figure IX F-10. The test results of the caisson compared to other suggested methods of temporary breakwaters are given in Table IX F-II.

TABLE IX F-2*

PARTICULARS OF WAVE BREAKER MODELS

| Principal Features | Wave-length: feet | Wave-height trough to crest: feet | | Remarks |
|---|---|---|---|---|
| | | Ahead of breaker | Behind breaker | |
| 1 Floating grids of brush-wood, logs, etc. | 160 | 6 to 10 | 6 to 10 | Grids moved with passage of wave and had no noticeable damping effect. |
| 2 Triangular section, both faces of brushwood. | 160 | 10.0 | 0.75 | Practically no wave motion behind breaker. Both sides of breaker were moving in and out, or "breathing," and there was some dis-turbance in consequence. |
| 3A Seaward face of brush-wood, rear face open at top, solid at bottom. | 160 | 10.0 | 2.5 | This breaker had consider-able damping effect, but was not so efficient as No. 2. (Fig. 3 (a)). |
| 3B As 3A, but with rear face solid to top | 160 | 10.0 | 0.3 | There was little movement behind the breaker--it was due to the structure "breath-ing" a little. |
| 4A Lower part solid, upper part open, with grids, on both faces. | 160 | 10.0 | 4.0 | The wave motion immediately behind breaker was much reduced, but further back the long swell appeared again, and reached a height of 6 to 7 feet. |
| 4B As 4A, with grids on rear face covered, and rear face open at bottom | 160 | 10.0 | 3.5 | Confused sea immediately behind breaker, re-forming into 6 to 7 foot waves again. |
| 5A Triangular supporting structure at rear, with vertical series of hori-zontal slots built up in front. | 160 | 10.0 | 5.0 | Wave reduced to about half-height, and confused sea caused by spilling effect. |
| 5B As 5A, but with forward face of triangle boarded in. | 160 | 10.0 | 2.0 | No waves could now pass through, since face was now solid. Waves came over top and caused small waves behind. |

* Reference IX 80 Todd, 1945

TABLE IX F-2 (Continued)

| Principal Features | Wave-length: feet | Wave-height trough to crest: feet | | Remarks |
|---|---|---|---|---|
| | | Ahead of breaker | Behind breaker | |
| 6A Simple triangular cross-section, completely submerged. | 160 | 10.0 | 9.0 | Wave crests tended to break just as they passed over breaker, but very little diminution of height. |
| 6B As 6A with vertical flap hinged to top edge of triangular section, the movement of flap being controlled by buoyant cylinders attached to it on rearward side. | 160 | 10.0 | 1.67 | Flap leaning backward, and water at rest 20 inches above bottom edge. The movement of flaps was restrained to a large extent by the presence of the cylinders behind, in the relatively calm water, and this small movement generated only small waves behind the breaker. Standing waves of some height were formed in front of the breaker due to reflexion. |
| 7A Solid face and back, with forward-curved parapet at top. | 160 | 10.0 | nil | Waves reflected from face, no disturbance behind. Breaking water at crests was thrown back and did not go over breaker. |
| 7B As 7A, but with circular holes in sea and rearward faces. | 160 | 10.0 | 0.25 | Water spilled through first row of holes above water, causing ripples behind breakwater. On face, large quantities of water were thrown out of holes when crest was next to breaker. |
| 8A Beach type with slots | 160 | 10.0 | 5.0 | Immediately behind breaker there was a confused sea. The waves re-formed and at 500 feet back there was a swell similar to that in front, and 5 to 7 feet high. |
| 8B As 8A, but with lower rows of slots filled in to give complete beach. | 160 | 10.0 | 3.0 increasing to 5 ft. aft of breaker. | Similar to 8A. |

TABEL IX F-2 (Continued)

| Principal Features | Wave-length: feet | Wave-height trough to crest: feet | | Remarks |
|---|---|---|---|---|
| | | Ahead of breaker | Behind breaker | |
| 9 Similar to 8 with equal slope each side | 160 | 10.0 | 3.0 | Waves reduced by breaker, but re-formed into swell again on moving away. |
| 10A Two vertical slotted walls | 160 | 10.0 | 7.0 | Waves reduced, but re-formed further on. |
| 10B As 10A, with interior of wall filled with brushwood. | 160 | 10.0 | 3.3 increasing aft of breaker. | Considerable reduction, but brushwood was evidently not packed tightly enough, and moved about between the walls. |
| 10C As 10B, with brush-wood more firmly packed. | 160 | 10.0 | Small to 6.7 ft. | Waves re-formed to about two-thirds of original height. |
| 11 Triangular base with vertical wall on top | 160 | 10.0 | 9.0 | Practically no reduc-tion in wave height. |

Towing and sinking tests were also performed on caissons and they were found to be easily towed and sunk in place. A small gap was left between caissons because it was felt that interlocking was too complicated and that the small gap was not large enough to diminish the effectiveness of the breakwater.

In Normandy a row of caissons was used seaward of the sunken ship breakwater. This type of sunken caisson breakwater is undoubtedly the best way to protect a temporary harbor from wave action. It affords protection independent of the wave height and period so long as the overturning, heaving and thrust of the wave is not beyond the design limits.

The invasion harbor of Normandy was destroyed by a storm larger than that for which the structures were designed. The caissons disintegrated due to the hydrostatic pressure of the water that fell into them as the waves crashed over the tops of the units. Twenty-five of the twenty-eight caissons of the nearly completed harbor at St. Laurent sur Mer were destroyed. The incomplete harbor at Arromanches was more protected and lost only five (Figure IX E-3). However, in later winter storms, the caissons were displaced through wave action, scour and slipping, and were further destroyed in a similar manner as at St. Laurent sur Mer. As proof of there sturdieness in 1951 Sweden refloated two of the remaining caissons to use for [illegible].

There is no proof available that interlocking, filling or more stable roofs would have improved the strength of the caisson breakwater at St. Laurent sur Mer, but this further construction should be made if time and material permit. However, experience has shown that extra caissons should be built and kept on hand to replace those that might be damaged.

2. Sunken Ships:

The sunken-ship breakwater is less stable and effective than the caisson type. The ship is sunk by opening sea cocks and by blasting holes in the bottom -- that is, by regular scuttle charges. Due to the shape of a ship's hull, it is less stable against tipping than the flat-bottom caisson. Since the shape of a ship's bow and stern allow more wave transmission than do the square ends of the caisson, overlapping is necessary when constructing a ship breakwater. While the construction of a ship is more sturdy than that of a caisson, ships - because of their length - will break their backs from heaving. This, in addition to scour and tipping, caused the failure of the ship breakwater at Normandy during the storm. However, like the caisson breakwater, a sunken ship breakwater gives good protection and is independent of the wave height and period as long as it is not damaged or topped by the waves.

3. Floating Breakwater:

During the Normandy Invasion, a floating breakwater was used as an outer breakwater where large ships could anchor outside the main harbor and discharge some cargo if possible or wait their turn at the piers. Figures IX F-11 and 12 show the breakwaters in place at St. Laurent sur Mer and Arromanches during the Normandy invasion. These floating breakwaters were very effective for waves less than 6 feet high and of short periods of up to 5 seconds. However, the effectiveness of this type of

breakwater is a function of the wave period, that is, for long wave periods a greater amount of wave energy is transmitted. This is the usual property of most wave-damping devices where only a partial amount of the wave energy is absorbed or reflected. The floating breakwater works on the principle that a semi-submerged body will reflect the waves, if its natural periods of oscillation are long compared to the wave period. To accomplish this feature, the floats for the Normandy Invasion were made in the shape of a cross and pratially filled with water. They were constructed on shore and easily towed to the site of the harbor. Figure IX F-13 shows a breakwater under construction. The major problem was in anchoring the structure, and it was found that not more than two sections could be successfully anchored together. The gaps thus caused between pairs were to be blocked by other staggered rows of the floating breakwater. The floating breakwaters used at Normandy were tested by model studies (See Figure IX F-14) and by phototype studies as well as by theoretical computations.

4. Underwater Breakwaters:

An underwater breakwater is a barrier constructed with its top at, or slightly above, the still-water level. Such a structure absorbs some of the wave energy by making the waves break prematurely. Some of the remaining wave energy is reflected, some is transmitted shoreward. Underwater breakwaters usually cost less to construct and maintain than conventional breakwaters (rubble mounds) which extend well above the highest expected tide stage. However, underwater breakwaters give less protection against wave action than other types, and are, therefore, seldom used in a permanent harbor development.

In localities where there is an appreciable tidal range, an underwater structure might be relatively ineffective during high tides. Moreover, during low tides, the top of the underwater structure might be above the still-water level, so that the structure would act like a conventional type breakwater. As such, it would have to withstand much greater wave forces than when submerged. For greatest effectiveness, therefore, underwater breakwaters should be used only in localities where the tidal range is relatively small.

a. Examples of Underwater Breakwaters:

In the known instances where underwater breakwaters have been constructed, they were built to help control beach erosion. They were constructed of sheet steel piling and placed parallel to the shoreline offhsore, with the top of the piling at about still-water level. Their use resulted in a reduction in the intensity of wave action on the shoreline, and consequently, a reduction in the rate of beach erosion. However, no data are available on any comprehensive or quantitative study of the effect of these structures on wave action.

An example of a natural underwater breakwater is the coral reef shown in Figure IX F-15. This shows, in a qualitative manner, the effectiveness of such a barrier in reducing the intensity of wave action on the lee side of the reef. Figures IX F-16, 17, 18, and 19 show examples of natural harbors formed by reefs.

An example of a stable underwater barrier is the outer 3,900 feet of the south jetty of the Columbia River. This rubble-mound jetty was unravelled by severe wave action until a concrete terminal was added to prevent further damage. The unravelled portion of the jetty was beaten down by the waves to a level 10 feet below the low water level. Information on the effectiveness of this underwater jetty has not been investigated.

As an example of underwater breakwaters used in an invasion harbor - sunken barges were used as an extension of the breakwater at Guam, but were displaced after being subjected to a typhoon.

b. Model Studies of Underwater Breakwaters:

For an invasion harbor on a relatively steep beach, sunken ships and barges have been suggested as underwater breakwaters. The results of experiments given below indicate the requirements of such breakwaters from the standpoint of reducing wave action. In addition, the important problems of the scour around and under, and also the anchorage of such sunken craft must be considered and solved. The most important requirement in the construction of a breakwater is that it must be firmly anchored and protected from scour of its foundation.

The most reliable quantitative data on the effect of underwater breakwaters on wave action are those obtained in model laboratory studies. In such studies, a wide variety of structure type has been investigated. The Beach Erosion Board (Reference IX-4) has reported on the damping effect of vertical walls and barriers with triangular and trapezoidal cross-section (Figure F IX-20). Tests on a trapezoidal section were also made in Switzerland by Stucky and Bonnard (Reference IX-79). More recently, Morison (Reference IX-60) has investigated wave action over rectangular barriers.

In all of these studies, the experiments were on two dimensional models, with the wave crests parallel to the barrier axis. The same depth water existed on the seaward and lee sides of the structures (with the exception of Morison's experiments, where rectangular breakwaters were placed on both a horizontal and a sloping bed (See Reference IX-61).

In addition, model studies have been made of wave action over various idealized reefs. As shown in Figure IX F-20, the reefs investigated included:

(i) an abrupt discontinuity followed to shoreward by a horizontal shelf

(ii) an abrupt discontinuity followed to shoreward by a uniformly shoaling bottom, and

(iii) an abrupt discontinuity followed by a horizontal shelf, a depression, and a continuation of the horizontal shelf.

These experiments were concerned primarily with the change in wave length after the waves passed the seaward edge of the reef. No information is given on the reduction in wave height after the waves passed over the reef. However, a theoretical study of the change in wave height (after waves have passed over a reef of type 2) has been made by Lamb (Reference IX-51).

Other theoretical work has been done on the following:

- wave action over submerged rectangular barriers, by Jefferys (Reference IX-45)
- submerged plane vertical barriers, by Dean (Reference IX-18); Ursell (Reference IX-82); and John (Reference IX-47)
- submerged plane horizontal barriers, by Heins (Reference IX-36)
- submerged circular cylinders, by Ursell (Reference IX-83); and
- the problem of reflections at inclined plane barriers has been analyzed by Dean (Reference IX-19).

University of California Model Studies on Rectangular Underwater Breakwaters

The following discussion summarizes the previously unpublished data of Morison (Reference IX-60), and compares the results whenever possible with theory as well as with the previous model studies on other types of Structures.

To determine the general effectiveness of rectangular underwater breakwaters in reducing wave action, a series of model studies was made in the one-foot wave channel in the Fluid Mechanics Laboratory at the University of California, Berkeley. The variables in these tests were height and width of the barrier, the depth of water, and the wave height and length. Tests were made with both a horizontal bottom and a beach with a slope of 19.2:1, (Figure IX F-21). The barriers were assembled by adding layer upon layer of plywood until their height reached the elevation of the still-water level. Barrier widths of 0.33, 0.67, and 1.00 feet were used in the tests with a horizontal bottom, and widths of 0.31 and 0.60 feet were used in the tests on the sloping beach. In these tests, four positions of the barrier were used in relation to the breaker position. These positions were 20, 30, 40, and 60 inches seaward of the breaker point. Shoreward from the barriers, a crushed rock beach was provided to absorb the waves which passed over the barrier. Approximately 70 runs, covering a large range of the variables, were made with a horizontal bottom, and about 10 runs were made with the sloping bottom.

Wave heights were measured for each barrier height, (shown in Figure IX F-21) by means of wire resistance elements. This recording system consisted of two pairs of wire resistance elements, each connected in series with an oscillograph and in parallel with separate transformers. The elements were mounted on point gages and calibrated by lowering them to successive positions in the water. During a test, only the fifth to tenth waves were recorded in order to minimize the possibility of reflection from the beach. This procedure also permitted the first few irregular waves to pass the recording

elements. During some tests, motion pictures of the waves action were taken with a 35 mm. camera.

In the tests with a horizontal bottom, the test procedure differed slightly from that employed with a sloping bottom. For the tests with a horizontal bottom a "steady-state" run was made first. This type of run consisted of first measuring wave heights by gages 1 and 2 (Figure IX F-21a) with no barrier in place. The average wave height so obtained was designated as the seaward wave height, $H_s$ (see list of symbols below). Also from the oscillograph record, the wave period and the time for the wave to travel from gage 1 to gage 2 was determined. From these latter data, the wave length was computed. The barrier next was placed in position (Figure IX F-21a) and the wave height was then measured at gage 2 after each increase in barrier height. In both the steady-state and in the test runs with the barrier in place, the wave height $H_i$, was taken as the average height of the fifth to tenth waves as measured on the oscillograph record.

In the experiments on the sloping beach, the resistance elements were located equidistant from the point where the waves broke when no barrier was in place (Figure IX F-21b). With no barrier in place, a steady-state run was made for each wave condition as discussed above in the tests on a horizontal bottom. From the oscillograph record, the height, $H_s$, was taken to be the height at gage 1. Also from the oscillograph record, the wave period and the travel time of the waves between gages 1 and 2 (Figure IX-21b) were determined, from which data the wave length near the breaker was computed. For each wave condition, barriers were placed at various multiples of the wave length seaward from the normal point of breaking. For a given barrier position, the barrier height was varied by adding successive layers of plywood. Wave height then was measured at gage 1 for each test condition, and the height, $H_i$, was taken as the average of the fifth to tenth waves passing the barrier.

With the above determined values (of water depth, barrier height and width, and wave characteristics), the values of the various dimensionless terms discussed below were computed for each run.

A theoretical study of waves passing over a submerged rectangular breakwater was made and presented as follows: the energy transmitted forward in a wave incident on a submerged barrier is partially transmitted and partially reflected. The transmitted energy is reformed into a wave motion subject to shoaling over the barrier. The wave length will decrease over the barrier and lengthen to the original value after leaving the barrier if the period is assumed to be constant. Cases are excluded of non-regular transition where the period changes abruptly at the edge of the barrier.

The energy transmitted forward with the incident wave motion per unit time and below a depth d-h is

$$E_R = -\int_d^{d-h} p \frac{\partial \phi}{\partial x}\, dy \qquad \text{IX F-1}$$

$$p = \rho\left(\frac{\delta \phi}{\delta t} - g\, y\right) \qquad \text{IX F-2}$$

$$= \rho \frac{g\, H}{2} \frac{\text{Cosh } k(y + d)}{\text{Cosh } k\, d} \text{Cos } (kx - \sigma t) \qquad \text{IX F-3}$$

where:

$$k = \frac{2\pi}{L} \qquad \text{IX F-4}$$

$$\sigma = \frac{2\pi}{T} \qquad \text{IX F-5}$$

The definition of symbols is given at the end of this study. Substituting and carrying out the integration, the time average of $E_R$ is:

$$\overline{E_R} = \frac{1}{16} g \rho H^2 C \frac{\text{Sinh } 2\, kh}{\text{Sinh } 2\, kd} \left(1 + \frac{2\, kh}{\text{Sinh } 2\, kh}\right) \qquad \text{IX F-6}$$

The total average energy transmitted per unit time in the incident wave is obtained by setting $h = d$ or

$$\overline{E_i} = \frac{1}{16} g \rho H_s{}^2 C \left(1 + \frac{2\, kd}{\text{Sinh } 2\, kd}\right) \qquad \text{IX F-7}$$

The average energy transmitted per unit time over the barrier is

$$\overline{E_T} = \frac{1}{16} g \rho H_s{}^2 C \left(1 + \frac{2\, kd}{\text{Sinh } 2\, kd}\right) \left[1 - \frac{\text{Sinh } 2\, kh}{\text{Sinh } 2\, kd} \frac{1 + \frac{2\, kh}{\text{Sinh } 2\, kh}}{1 + \frac{2\, kd}{\text{Sinh } 2\, kd}}\right] \qquad \text{IX F-8}$$

There are two cases to be considered here, if the wave period is assumed to remain constant. The wave length and wave velocity depend upon the depth according to the relations:

$$k = \frac{\sigma^2}{g} \text{Coth } kd \qquad \text{IX F-9}$$

$$C = \frac{g}{\sigma} \text{Tanh } kd \qquad \text{IX F-10}$$

Case 1. The transmitted wave is measured in water of depth d shoreward of the barrier. In this case k and C are equal to their original values for the incident wave. Since $\overline{E_T}$ must have the same form as $\overline{E_i}$ the transmission coefficient is:

$$\left(\frac{H_i}{H_s}\right) = \sqrt{1 - \frac{\text{Sinh } 2\,kh}{\text{Sinh } 2\,kd}\,\frac{n\,(h)}{n\,(d)}} \qquad \text{IX F-11}$$

where:

$$n\,(h) = \tfrac{1}{2}\left(1 + \frac{2\,kh}{\text{Sinh } 2\,kh}\right) \qquad \text{IX F-12}$$

Case 2. The transmitted wave is measured over the barrier as for a reef of type 1. The transmitted wave is subject to shoaling over the barrier, its height and length varying in such a way that energy and wave period are conserved. The average energy transmitted per unit time over the barrier can be written in the form:

$$\overline{E_T} = \frac{1}{16}\,g\,\rho\,H_i^2\,C_a\left(1 + \frac{2\,k_a\,a}{\text{Sinh } 2\,k_a\,a}\right) \qquad \text{IX F-13}$$

where the subscript a refers to conditions in water of depth a = d-h. Comparing Equation 1 x F-8 and 13 we find for the transmission coefficient:

$$\left(\frac{H_i}{H_s}\right)_a = S\left(\frac{H_i}{H_s}\right)_d \qquad \text{IX F-14}$$

$$S = \text{shoaling factor} = \sqrt{\frac{C_d\,n_d\,(d)}{C_a\,n_a\,(a)}} \qquad \text{IX F-15}$$

The shoaling factor can be readily determined from Wiegel's Study (Reference IX-86) which gives the relative wave height in a wave moving from depth d to depth d-h without reflection.

Two important cases merit individual attention. For shallow water ($kd \ll 1$) one finds

$$\left(\frac{H_i}{H_s}\right)_d \cong \sqrt{1 - \frac{h}{d}} \qquad \text{IX F-16}$$

$$\left(\frac{H_i}{H_s}\right)_a \cong \sqrt[4]{1 - \frac{h}{d}} \qquad \text{IX F-16}$$

For deep water ($kd \gg 1$) one finds,

$$\left(\frac{H_i}{H_s}\right)_d \cong \sqrt{1 - 4\,e^{-2kd}\,(\text{Sinh } 2\,kh)\,n\,(h)} \qquad \text{IX F-17}$$

which is nearly equal to unity unless the barrier is near the free surface in which case

$$\left(\frac{H_i}{H_s}\right)_d \cong \sqrt{1 - e^{-2ka}} \qquad \text{IX F-18}$$

For shallow water (and for deep water if the barrier is near the surface,) the transmission coefficients are the same as if it is assumed initially that a fixed fraction of the total energy associated with the incident wave is reflected.

The symbols used have the definitions:

a = depth of barrier below still-water level

B = width of sloping barrier

d = depth of water

g = acceleration of gravity

$H_i$ = wave height inshore from barrier

$H_s$ = wave height seaward from barrier

$H_o$ = wave height in deep water

h = height of barrier

L = wave length

$L_s$ = wave length seaward from barrier

$L_b$ = wave length at breaker

$L_o$ = wave length in deep water

s = beach slope

T = wave period

W = width of barrier

$\theta$ = angle of inclination of sloping barrier

As described above, the wave characteristics, barrier dimensions, etc., (as illustrated in Figure IX F-21) were measured during each run. A consideration of the various terms shows that the following dimensionless groupings can be used to represent the relationship between the transmission coefficient $H_i/H_s$, and the other variables, i.e.,

$$H_i/H_s = f\ (H_s/L_s,\ W/L_s,\ h/d,\ d/L_s) \quad \text{IX F-19}$$

To demonstrate the relative importance of these variables, certain of the data have been summarized in Figures IX F-22 to 25 inclusive. Figure IX F-22 shows the general effect of the wave steepness, $H_s/L_s$, on the transmission coefficient, $H_i/H_s$, for various $d/L_s$, and relative barrier width, $W/L_s$. The

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two sets of experimental data shown in this Figure indicate that a barrier of a given relative height, h/d, is more effective in damping relatively steep waves than relatively flat waves. This is to be expected, since a wave which is near the critical steepness for breaking can be easily "tripped" and made to break by any outside disturbance. It should be noted that this effect of steepness becomes relatively small when the top of the barrier is at, or slightly above, the still-water level. This conclusion was also established theoretically by Jefferys (Reference IX-45) for waves of small amplitude in shallow water. The theoretical relationship of Jefferys is plotted in Figure IX F-26, with the transmission coefficient, $H_i/H_s$, shown as a function of relative barrier height, h/d, with various selected values of relative width, $W/L_s$, shown as a parameter. The curves in Figure IX F-26 present the interesting conclusion that the most effective damping action for a particular barrier height occurs when the width of the barrier is 1/4, 3/4, 5/4 times the $\sqrt{1 - h/d}$ ... of the wave length. To compare Jefferys' theoretical relationship with the experimental data in Figure IX F-22, the theoretical curve for the condition when $W/L_s = 0.10$ is shown. For the smaller amplitude wave (low wave steepness) there is fairly good agreement between theory and experiment. For the steeper wave, however, the assumptions underlying the theoretical treatment no longer apply and theory and experiment differ greatly.

In a presentation similar to Figure IX F-22, Figure IX F-23 shows the effect of relative depth, $d/L_s$, on the transmission coefficient for two conditions where the wave steepness and relative dimensions of the barrier are approximately equal. The data in this Figure indicate that a greater damping effect is obtained when the relative depth, $d/L_s$, becomes small. This conclusion appears reasonable and might be predicted qualitatively if it is recognized that a larger percent of the wave energy is concentrated in the upper layers of the depth for large $d/L_s$ values than for low values of $d/L_s$. Hence, this energy can "spill over" the top of the barrier more easily than when the energy is more uniformly distributed throughout the depth as in the case of waves with small values of $d/L_s$. Figure IX F-23 shows that once the top of the barrier is located near still-water level, the effect of $d/L_s$ on the transmission coefficient becomes unimportant. (Similar to the case in Figure IX F-22) Again for comparative purposes, the theoretical curve of Jefferys (Reference IX-45) for a value of $W/L_s = 0.10$ is presented. Also shown are curves as obtained by the theory presented above.

In connection with the "spill over" effect mentioned above, it is interesting to note that by estimating the reflected wave energy, and assuming conservation of both energy and period, one is led directly to simple formulae for the transmission coefficients. As a first approximation, the energy transmitted forward with the incident wave per unit time and below depth d-h is assumed to be totally reflected. Calculations show that the reflection decreases the wave height, while the shoaling action over the barrier increases the wave height and decreases the wave length. The net effect is



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that the transmission coefficient ranges from 1 to 0 as h/d ranges from 0 to 1. As to be expected, the transmission coefficient drops rather sharply in going shoreward from the barrier. The wave length increases to its original value. The agreement with experiment shown in Figure IX F-23 is good. A comparison of the theoretical curves with the experimental points in Figure IX F-23 indicates that as might be expected the theory of Jefferys is seriously in error for relatively steep waves.

As predicted theoretically and illustrated in Figure IX F-26, an important variable determining the effectiveness of an underwater breakwater is the relative width, $W/L_s$, of the structure. Experimental data on this factor is shown in Figure IX F-24, where the transmission coefficient is plotted against the relative height (as in Figures IX F-22 and 23) with the relative width as a parameter. The plot in Figure IX F-24a pertains to a wave of relatively low steepness (and also with a low $d/L_s$ value); whereas, Figure IX F-24b pertains to a wave with a relatively high steepness (and also with a high $d/L_s$ value). Also shown in Figure IX F-24 for comparative purposes, are theoretical curves of Jefferys (Reference IX-45) for relative barrier width, $W/L_s$, of 0.25 and 0.10 in Figure IX F-24a, and 0.25 and 0.12 in Figure IX F-24b. The data shown in Figure IX F-24a tends to conform to the theoretical relationship of Jefferys (Reference IX-45) (Figure IX F-26), since the experimental conditions closely approximate the theoretical assumptions of small amplitude waves in shallow water. It is interesting to observe that the theoretical curve for $W/L_s = 0.10$ practically coincides with the experimental points for the barrier of the same relative dimensions.

With respect to Figure IX F-24b (which presents the results of tests with a high wave steepness), it is noted that the transmission coefficient appears to be more sensitive to barrier width than was the case presented in Figure IX F-24a; however, it must be recognized that the barriers had relatively large widths in the runs presented in Figure IX F-24b as compared to those in Figure IX F-24a. As might be expected, the data shown in Figure IX F-24b indicate that the wider the barrier (higher values of $W/L_s$), the better is the damping action. Comparison of the experimental date with the theoretical curves of Jefferys (Reference IX-45) show comparatively good agreement for values of $W/L_s = 0.12$ and $W/L_s = 0.24$. Poor agreement is observed, however, between theory and experiment for $W/L_s = 0.36$. The experimental data show that a greater degree of damping can be obtained with the wider barriers than the theoretical maximum shown on Figure IX F-26. The fact that the wave in Figure IX F-24b was not a wave of small amplitude probably explains the divergence between theory and experiment.

Other information of interest is the effect of barrier position on wave damping. In this respect "barrier position" pertains to the distance of the barrier from the point of breaking when no barrier was in place. The tests to yield information on this phase of the problem were conducted on a uniformly sloping solid beach (19.2:1) where the waves actually broke on the test beach. The method of conducting these tests is described above in connection with Figure IX F-21b. The additional parameter in these tests, compared to those described above for a horizontal bed, was the location of the barrier. This location was expressed as a multiple (or fraction) of the wave length as measured in the surf zone.

Figure IX F-25 shows typical results from these tests wherein the transmission coefficient is plotted against relative barrier height, with barrier position as a parameter. The data shown in Figure IX F-25 were obtained with a barrier width of about one-sixth of the wave length. Since the measurements were made in or near the breaker zone, the wave steepness was up to the limiting value. For the conditions summarized in Figure IX F-25, it appears that the transmission coefficient, in general, is relatively insensitive to barrier position. As the larger relative barrier heights, h/d, are approached, however, a slightly better damping action appears to result if the barrier is located at or near one wave length from the breaker line. This would indicate that waves could be effectively "tripped" by an underwater breakwater and made to break and lose much of their energy if the barriers were located a wave length or more seaward from the normal breaking point. For the sloping beach, as well as for a horizontal bottom, the most effective damping action of an underwater breakwater occurs when the top of the structure is placed at the still-water level. On natural beaches, the bottom usually can be assumed to be horizontal over a distance of a wave length or more. Therefore, the various data presented for tests on a horizontal bottom can be considered to apply with reasonable accuracy for most natural beaches.

As mentioned above, previous model studies have been conducted on underwater breakwaters with triangular and trapezoidal cross-sections as well as on barriers which consisted of a thin vertical wall (Figure IX F-20). These data have been summarized by the Beach Erosion Board (Reference IX-4). However, as that analysis did not consider all of the dimensionless factors discussed in the above section, the Beach Erosion Board data were re-analyzed to conform to the method as discussed above, of presenting the data for rectangular barriers. As the number of runs for each barrier shape tested by the Beach Erosion Board was relatively small; it was necessary to cross-plot the original data to permit a presentation of the type of curves given above. Figure IX F-27 shows the transmission coefficient as a function of relative barrier weight and wave steepness for the triangular and the trapezoidal sections tested by the Beach Erosion Board (Figures IX 20a-20c). Also shown on these plots are the average curves as presented by the Beach Erosion Board (Reference IX-4) where the effect of the wave steepness was omitted. The wide scatter of points in the original plots of the Beach Erosion Board, particularly for the trapezoidal section, probably was due to the effect of wave steepness. The cross-plots of the original Beach Erosion Board data show the effect of wave steepness (Figure IX F-27). This effect is relatively great in the case of the trapezoidal breakwater (Figure IX F-27a), but appears relatively small in the case of the triangular section (Figure IX F-27b). In the two plots shown in Figure IX F-27 it is interesting to note that with the trapezoidal and triangular barriers, a greater degree of damping was observed for the lower wave steepnesses than for the steeper waves. The opposite effect was noted, however, for the rectangular barrier (Figure IX F-22).

Examination of the Beach Erosion Board data shows that, for waves of low steepness the relative depth, $d/L_s$, was also small. Therefore, (as might be expected from a study of Figure IX F-23) the relative position of the curves shown in Figure IX F-27 may be affected more by the relative depth than by the wave steepness. The Board's data were too meager, however, to permit a separation of the effects of these two factors.

Because of the small amount of data for the vertical wall, no attempt is made here to summarize these results in curve form. Interesting theoretical treatments of vertical walls are those by Dean, John and Ursell, (References IX 18, 47, and 82). These give the transmission coefficient for the two types of fixed vertical barriers shown in Figure IX F-28a. Figure IX F-29 gives plots of both transmission and reflection coefficients for these two types of barriers, as determined by Dean and Ursell (References IX 18 and 82). The reflection coefficient in the above instance is defined as the ratio of the wave heights (at a great distance from the barrier) of the reflected and incident waves.

In connection with wave action over an underwater reef, this problem has been approached from both the theoretical and the experimental standpoint. Lamb (Reference IX-51) has considered theoretically the case of wave action over a reef (Figure IX F-28) with shallow water in front of the barrier. In this case, expressions for both the transmission coefficient and the reflection coefficient were determined. These coefficients are shown in Figure IX F-27 as a function of the relative height of the reef. This Figure also shows curves of the transmission coefficient as plotted from Lamb's theory. The two curves shown pertain to the cases where $d/L_s$ approaches zero and $d/L_s \cong 0.33$. The theoretical results are totally different. No experimental data are available for comparison in this situation.

The only known experimental work approximating the theoretical treatment of Lamb (Reference IX-51) is that of the Beach Erosion Board (Reference IX-5). The various reef arrangements that they tested are shown in Figures IX 20e and 20 g. The primary concern of these investigations was the change in wave length after waves passed over a reef. They gave no information on the transmission coefficient. Of interest in these tests, however, was the establishment of a criterion for determining whether a wave would pass over a reef as a single wave, or break into a multiplicity of waves, as illustrated in Figure IX F-15. This criterion for a regular transition is that when the value of the term $\sqrt{H_o L_o}/a$ is less than 2.0 the transition is regular. For larger values of this term, a wave passing the reef will break into a number of waves.

Experiments have been made by Hamilton (Reference IX-31) on the damping action of plane-sloping surfaces which extend below the still-water level only a portion of the total depth. In such an instance, some wave energy is transmitted to the lee side by the waves passing beneath the structure. The upper edge of the plane surface in these tests was located above, at, and below the still-water level. The transmission coefficient was found to be a function of the relative width of the surface, $L_s/B$, with the angle of inclination of the surface being a parameter (Figure IX F-31). From the scatter of the points in this Figure, it appears that perhaps the wave steepness and relative depth should also have been considered as parameters. As may be seen from Figure IX F-31 the submerged plane was relatively ineffective in damping the waves; whereas, when the upper edge of the plane was at or above the still-water level (Figures IX F-31a and 31b)it was effective in damping the waves.

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c. Conclusions:

Within the limits of the experiments on rectangular underwater barriers, the following conclusion can be stated:

(i) For a given relative barrier dimension and relative depth, the transmission coefficient is greater (small damping action) for relatively flat waves than for relatively steep waves. Also for a given relative barrier dimension and wave steepness, the transmission coefficient is greatest for large relative depths.

(ii) A relatively wide barrier has a better damping action (low transmission coefficient) than a relatively narrow barrier, especially for relatively steep waves.

5. Air Breakwaters:

in 1907, Mr. Philip Brasher (Reference IX 8, 9, 11, and 74) suggested and patented the idea of an air breakwater. His method was to pump large volumes of air at a pressure only slightly greater than the hydrostatic pressure due to the depth of the water. The air was to be pumped through closely spaced holes in a submerged pipe line. The curtain of air would then reduce the mass of water and cause opposing currents which could not support wave actions of any appreciable amplitude.

The Brasher Breakwater Company was formed and claimed successful operation at the Standard Oil Pier at El Segundo in 1915; in the salvage of U. S. S. Lanhec in 1905; at the loading pier of a quarry at Crotch Island, Maine; and in protection of the Million Dollar Pier, Atlantic City. The set-up at El Segundo is shown in Figure IX F-32. Other experiments did not claim success. These were performed by the English in 1925; the Submarine Squadron IV at Pearl Harbor in 1937; I.A. Bogolepoff, Leningrad USSR 1937; (Reference IX-28) Humble Oil Co. at Creole, Louisiana in 1948; and by John H. Carr, California Institute of Technology, Pasadena California, in 1949 (Reference IX-13). The conclusion drawn by all was that the air breakwater damped only surface chop, or very short-period deep-water waves.

Bogolepoff and Carr did many experiments, and concluded that the air breakwater was impractical because so much power would be required to supply the necessary compressed air to damp all waves in shallow water. Mr. Bogolepoff gives the example of a wave height of 4.0 m, and length of 27.0 m and a wave period of 4.2 seconds. The quantity of air necessary to completely eliminate this wave would be 300 $m^3$/min./m at 4 atms and 15° C. which for a breakwater 100 m long requires a compressor plant of about 50,000 $in.^3$/min. which would require about 240,000 h.p. Carr computed that 4,000 CFM/ft. or about 260 h.p/ft. is necessary to stop deep-water waves of 300 feet length (7.7 sec. period) with the air release 100 feet below water surface. These results are considerably different from the 10 CFM/ft. claimed



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by Brasher. Figure IX F-33 and Table IX F-V below show some of the results obtained by Carr.

The air breakwater will dampen wind chop and very short period waves, so that if considerable pumping equipment is available where this type of waves is giving trouble, the construction of the pipeline may be of considerable value as a temporary breakwater. The best results obtained in the past were with a single row of pipe with the holes pointed downward.

TABLE IX-F-V
(From Reference IX-13, Carr)

| Q<br>Cfm/ft | L<br>ft | T<br>sec | HP<br>ft |
|---|---|---|---|
| 300 | 100 | 4.4 | 20 |
| 4,000 | 300 | 7.7 | 260 |
| 24,000 | 600 | 10.8 | 1575 |

6. Rubble-Mound Breakwater:

A rubble-mound breakwater consists of a dumped core of small rock which is sometimes graded with an approximate maximum weight of 50 lbs/stone. Cap, facing and armor rock are placed over the core in as many as three layers. This stone weighs from 2 to 15 or more tons each. For some breakwaters, stone of 25 tons or more are used; if still heavier material is needed, then precast concrete blocks are used. The rock should be dense, and of such quality that it will not disintegrate in salt water and has good resistance to abrasion. The core may be dumped from floating equipment by bottom dump barges, by bulldozing the rock of the barge or by conveyor belt barges. A trestle for a railroad may be built and the core dumped from the side of this. Figure IX F-34 is an example of this type of construction. Another method is to build a roadway (with turn-arounds) on the top of the mound and dump the rubble off the end of the mound by trucks. This method was used on Guam. The distance and type of transportation between the breakwater and the quarry greatly influence the method of construction. The severity of the general yearly wave history influences the possibility of the use of floating equipment, and at times the trestle type construction. The armor rock is placed with a minimum of voids by use of cranes, floating railroad on mobile ground crane. The side slope is generally 1 to 1½, 1-3/4 and 2. A unique type of breakwater seawall combination is shown in Figure IX F-35. A rubble mound breakwater can be built in any depth of water and nearly all types of foundation where allowance is made for settling. Figure IX F-36 is a typical structure. A rubble mound requires a large amount of material and continuous maintenance. It is not suitable for a wharf. To prevent the end from unravelling, a concrete terminus may be necessary. Figure IX D-3 shows the massive terminus used on one Columbia River Jetty.

If quarrying the stone, (for a rubble mound breakwater) the charges should be set to obtain the largest size armor stone. This stone is stock-piled and the quarry waste is used for the core material. The calculations for the size of stone that a wave will move and the best slope for a rubble mound are discussed in Section IV of this manual.

7. Wall Breakwater:

Wall breakwaters must be placed on a solid foundation that will not settle appreciably. This type of breakwater can be built in water up to 65 feet deep (see Figures IX D-2 and IX F-37). The walls may be vertical or sloping. When a wall is placed on a rubble mound, it is generally constructed of blocks, of concrete caps, caissons (up to 35 feet), or timber cribs (up to 40 feet). Figure IX F-38 shows several types of concrete caps on rubble mounds. The construction of a block-type wall is less expensive than a complete rubble mound if suitable rock can be obtained. The block and caisson type construction is keyed and dowled together. The blocks are sometimes placed on an angle as shown in Figure IX F-39. Piano blocks are sometimes used, Figure IX F-40. Blocks are mortared together while caisson and timber cribs are filled with rock and concrete. Figure IX F-41 shows a filled caisson and Figure IX F-42 shows a filled timber crib. Solid concrete caps are used. When large caissons are used, they are floated to the site and sunk into place. Figure IX F-43 shows the details of a caisson. It should be remembered in construction that concrete should not be dropped vertically more than 5 feet.

Wall breakwaters can be made by driving sheet steel piling, concrete sheet piling, and timber piling in depths up to 40 feet. Figures IX F-44 to 47 show several types of this method. The steel and timber pilings are not usable for long periods in salt water. Steel should have a protective covering (asphalt chromate emulsion) and timber should be creosoted. Concrete should be made with high silica cement. A combination of several types of caisson and sheet pile is shown in Figure IX F-48.

Wall type breakwaters are considered by some authorities to be as good as the rubble mound, less expensive, and less time-consuming to build. If built properly, the wall type breakwater requires less maintenance. The wall type breakwater should not be built in a depth of water where waves will break on it during severe storms. It is most effective where it reflects the unbroken waves. Calculation of the wave force onwalls is given in Section IV of this manual.



FIG. IXF-1 - Sunken caissons breakwater in place at Arromanches (89)

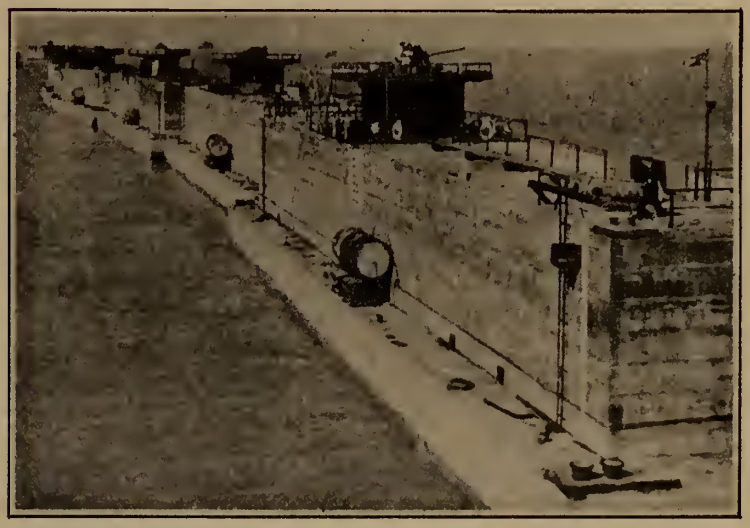


FIG. IXF-2 - Complete units, showing guns and anchors (89)

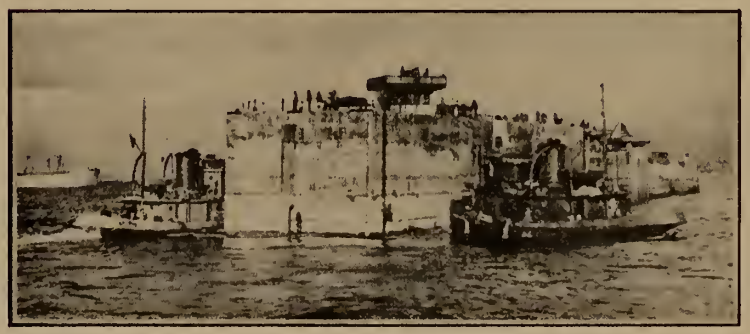


FIG. IXF-3 - Caisson units (Phoenixes) being positioned and sunk to form breakwater (53)

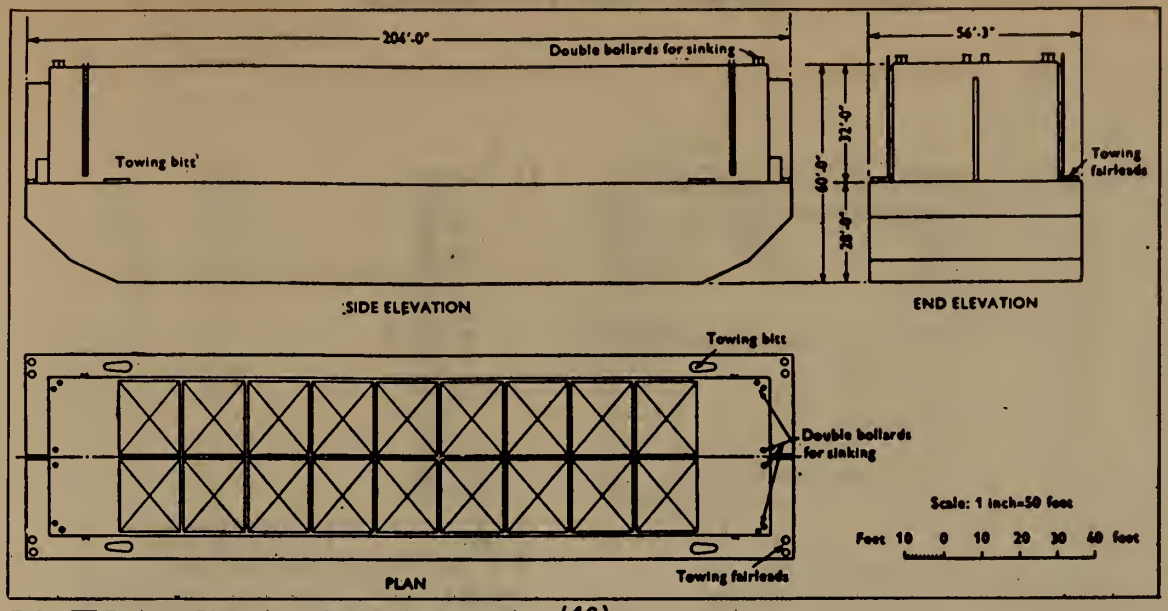



FIG. DXF-4 - Phoenix caisson, largest size (46)

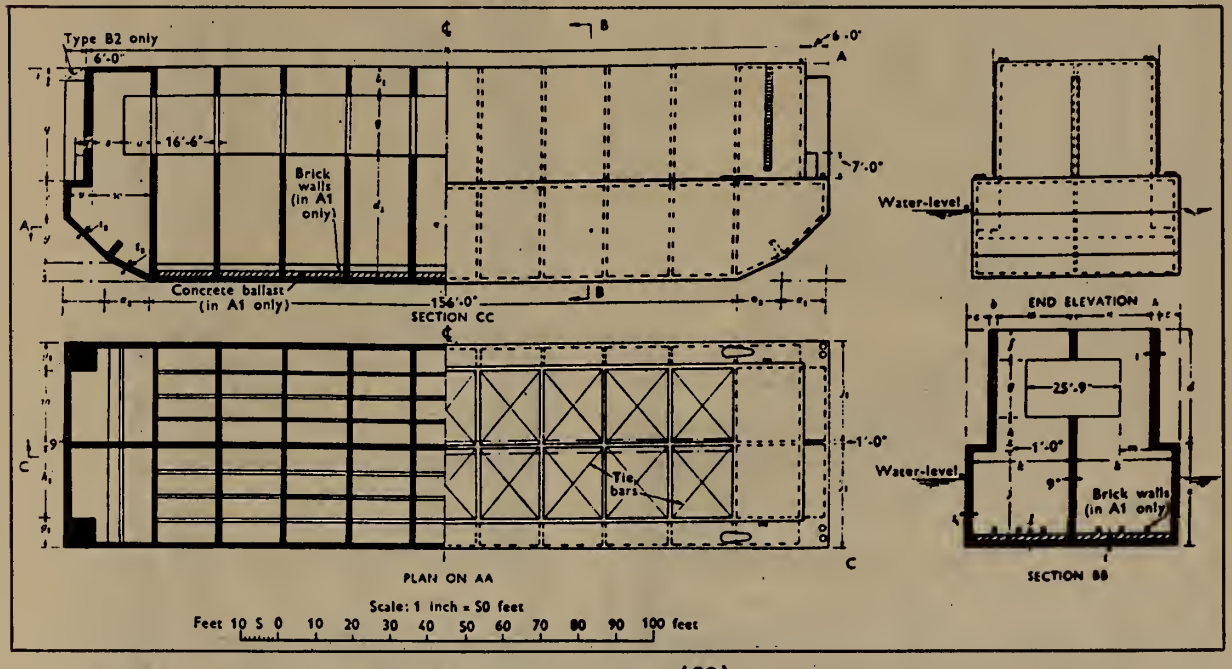



FIG. DXF-5 - Principal dimensions of Phoenix units (89)

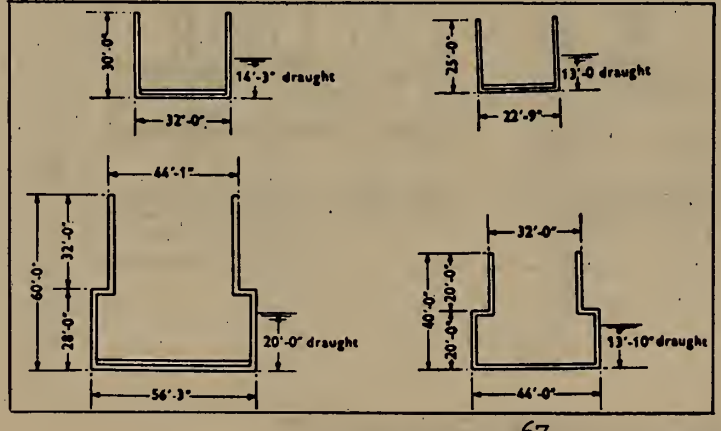



FIG. DXF-6 - Cross-section dimensions of various Phoenixes (46)

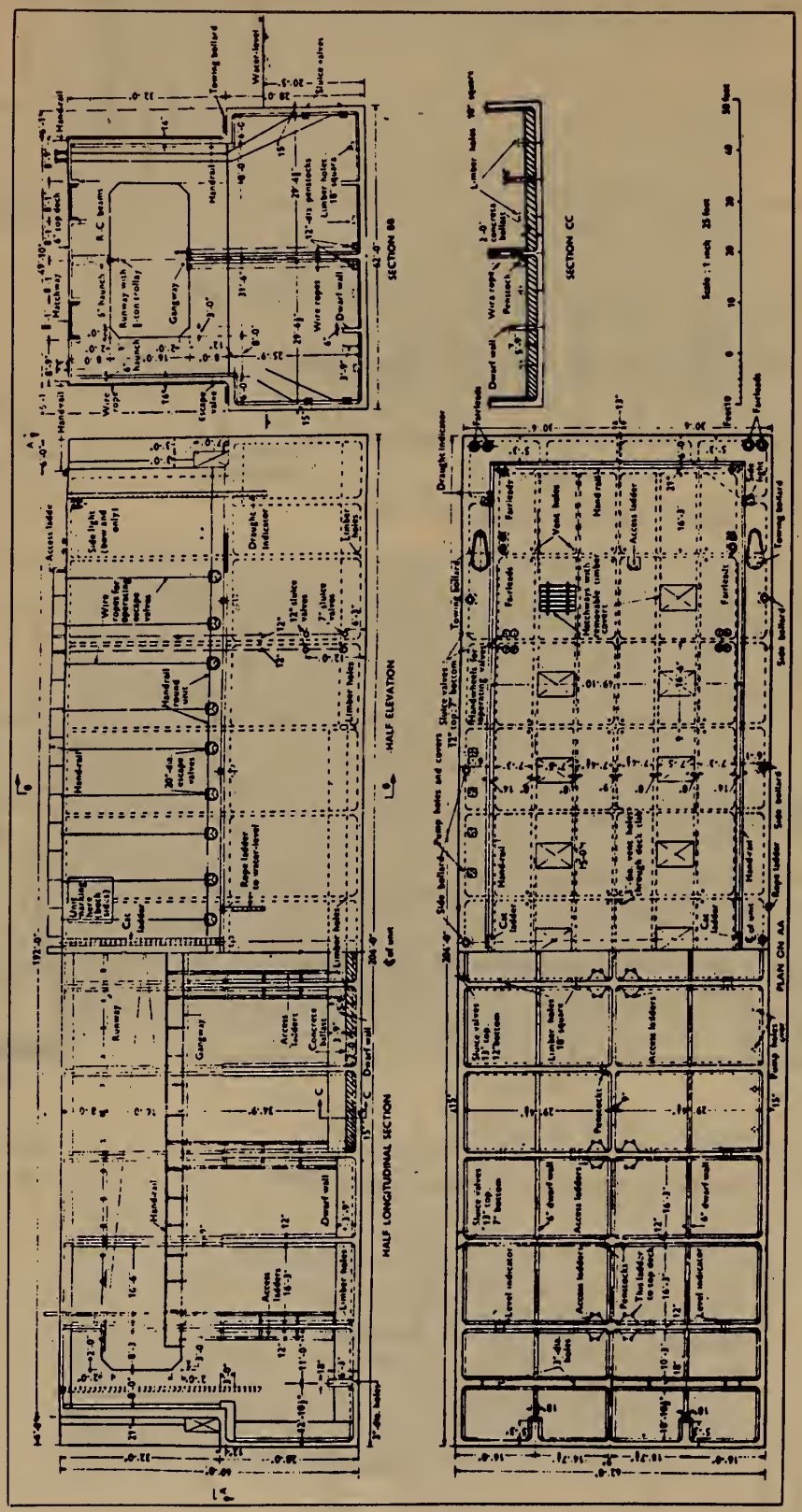


FIG. IXF-7 - Phoenix unit type *A1*, general arrangement (60)

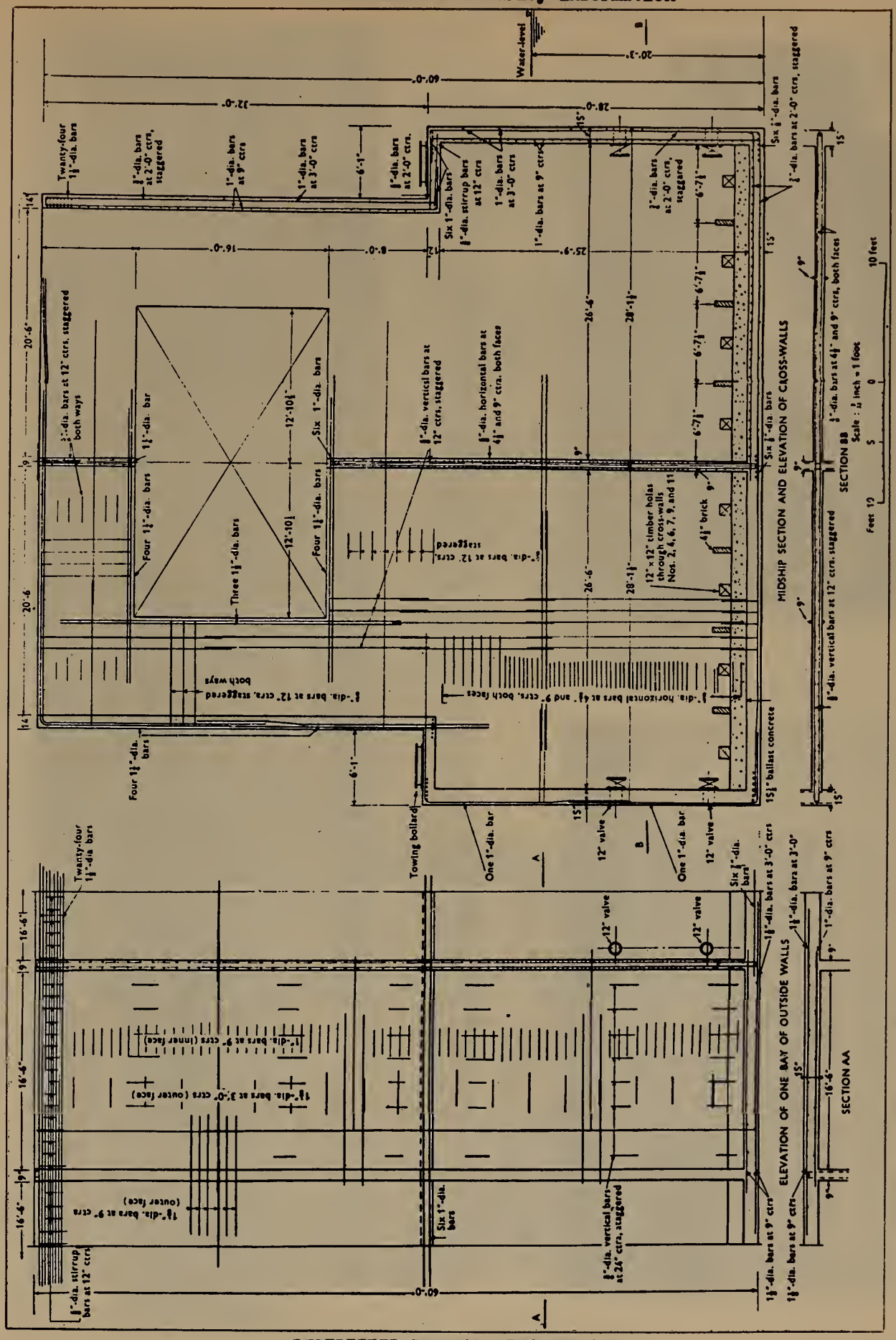



FIG. IX F-8 - Typical reinforcement details: unit type A1 (89)



FIG. IX F-10 - Model test of caissons (80)

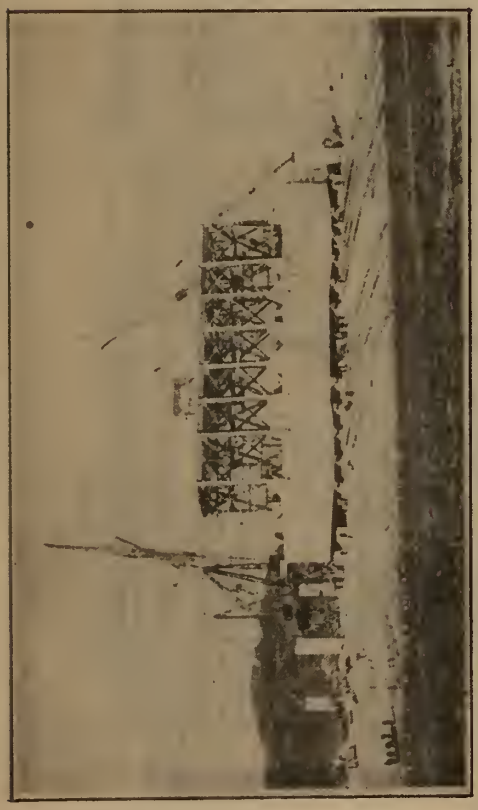


FIG. IX F-9 - Caisson before launching (43)



FIG. IX F-11 - Floating breakwater (Bombardon) at St. Laurent (52)



FIG. IX F-12 - Floating breakwater (Bombardon) at Arromanches (52)

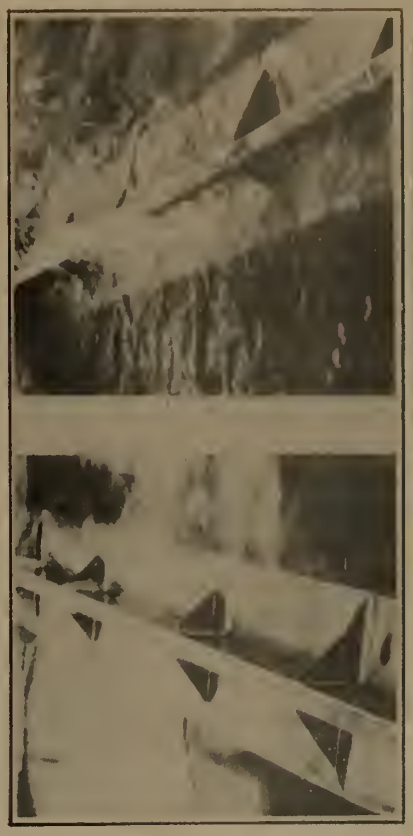


FIG. IX F-14 - Model test of floating breakwater (80)



FIG. IX F-13 - Floating breakwater (Bombardon) under construction (52)

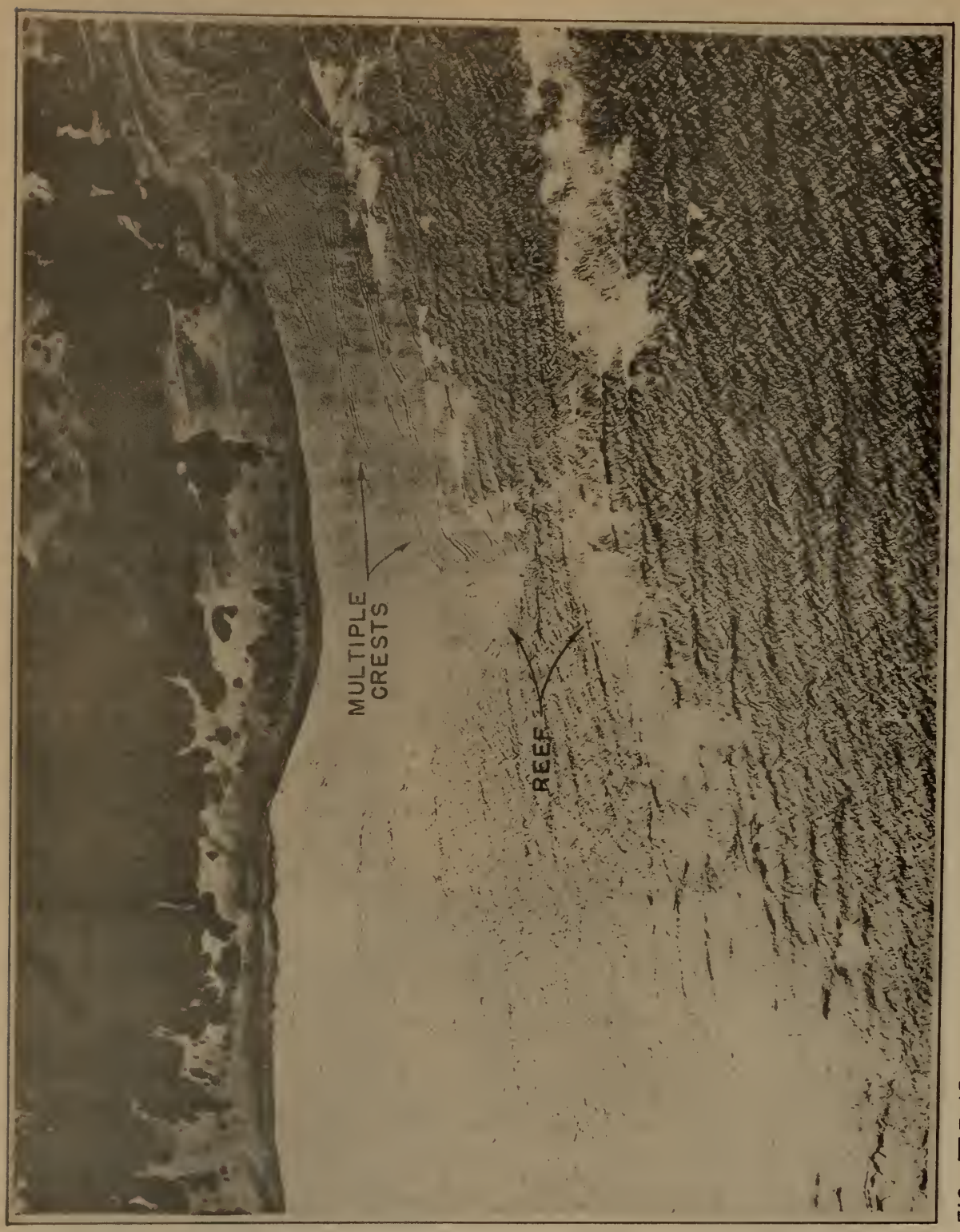



FIG. IX F-15 - Photograph of wave action on natural reefs. Note the damping action as the waves pass over the successive reefs. (48)




FIG. IXF-16 - Cocos-Keeling Atoll, Indian Ocean (77)




FIG. IXF-17 - Truk Barrier, Caroline Islands (77)




FIG. IXF-18 - Ngau, Fiji Group (77)




FIG. IXF-19 - Wake Atoll (77)

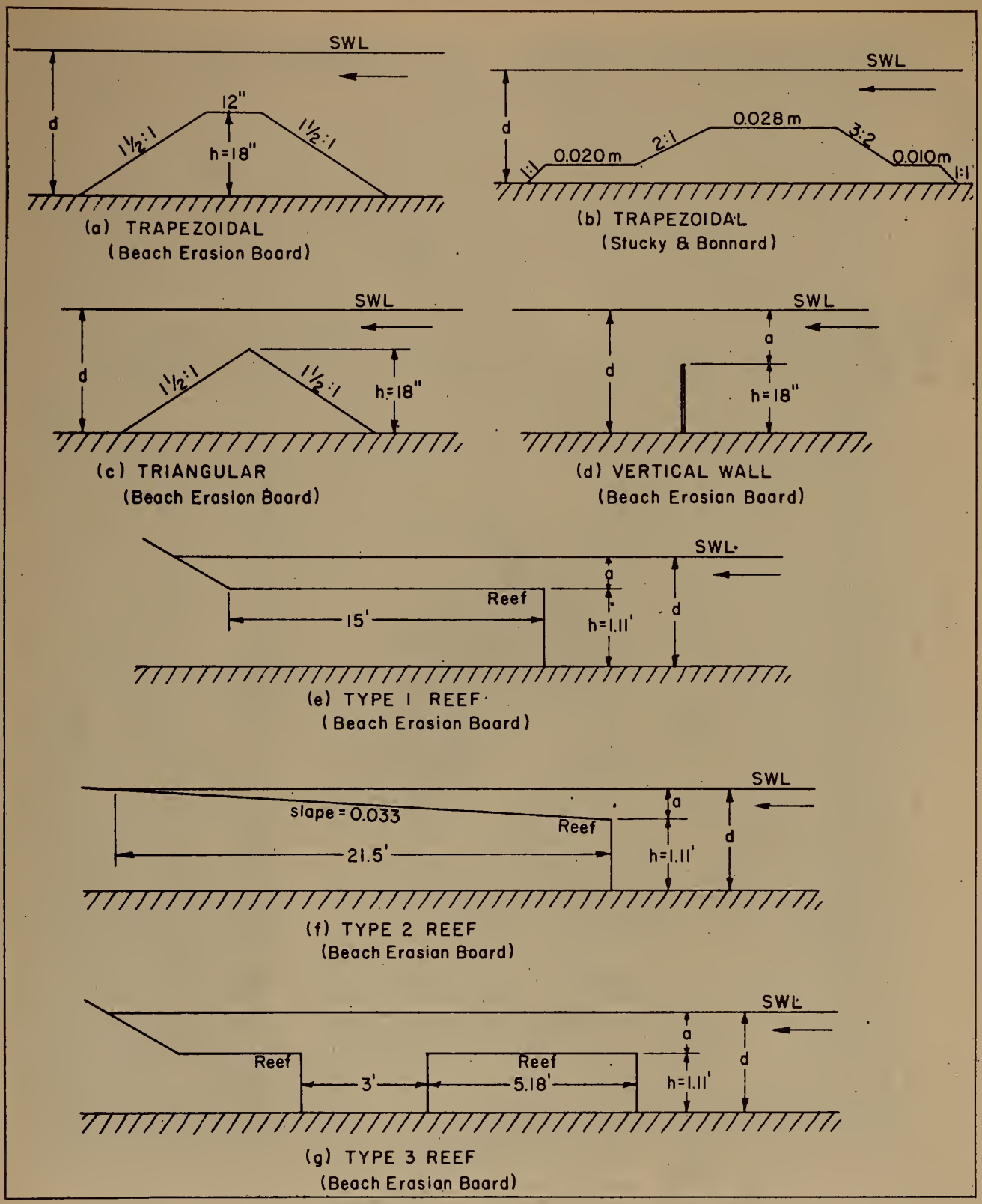



FIG. IXF-20 - Various types of underwater barriers investigated by model studies [48]

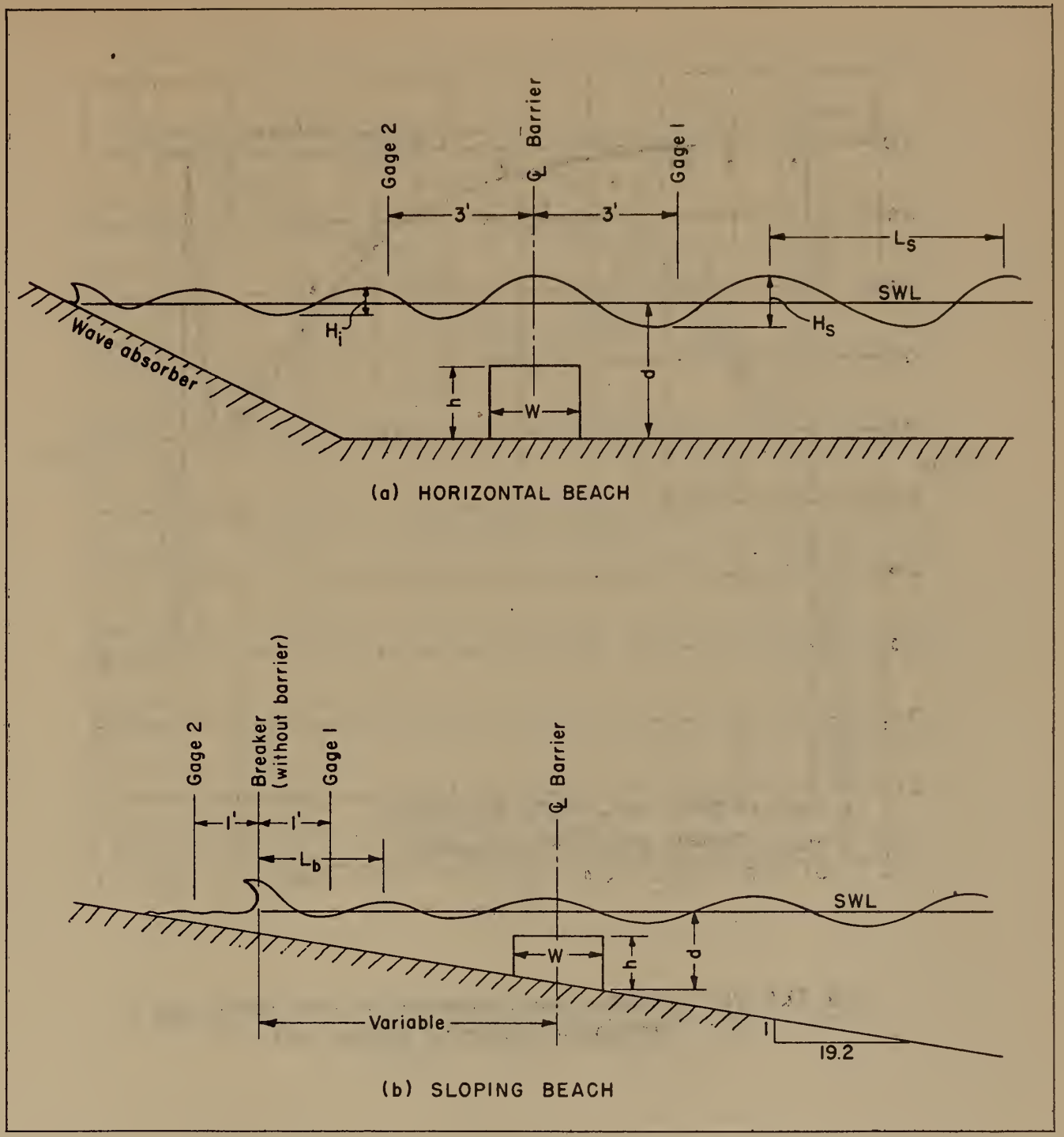



FIG. IXF-21 - Experimental equipment for tests on submerged rectangular barriers [48]

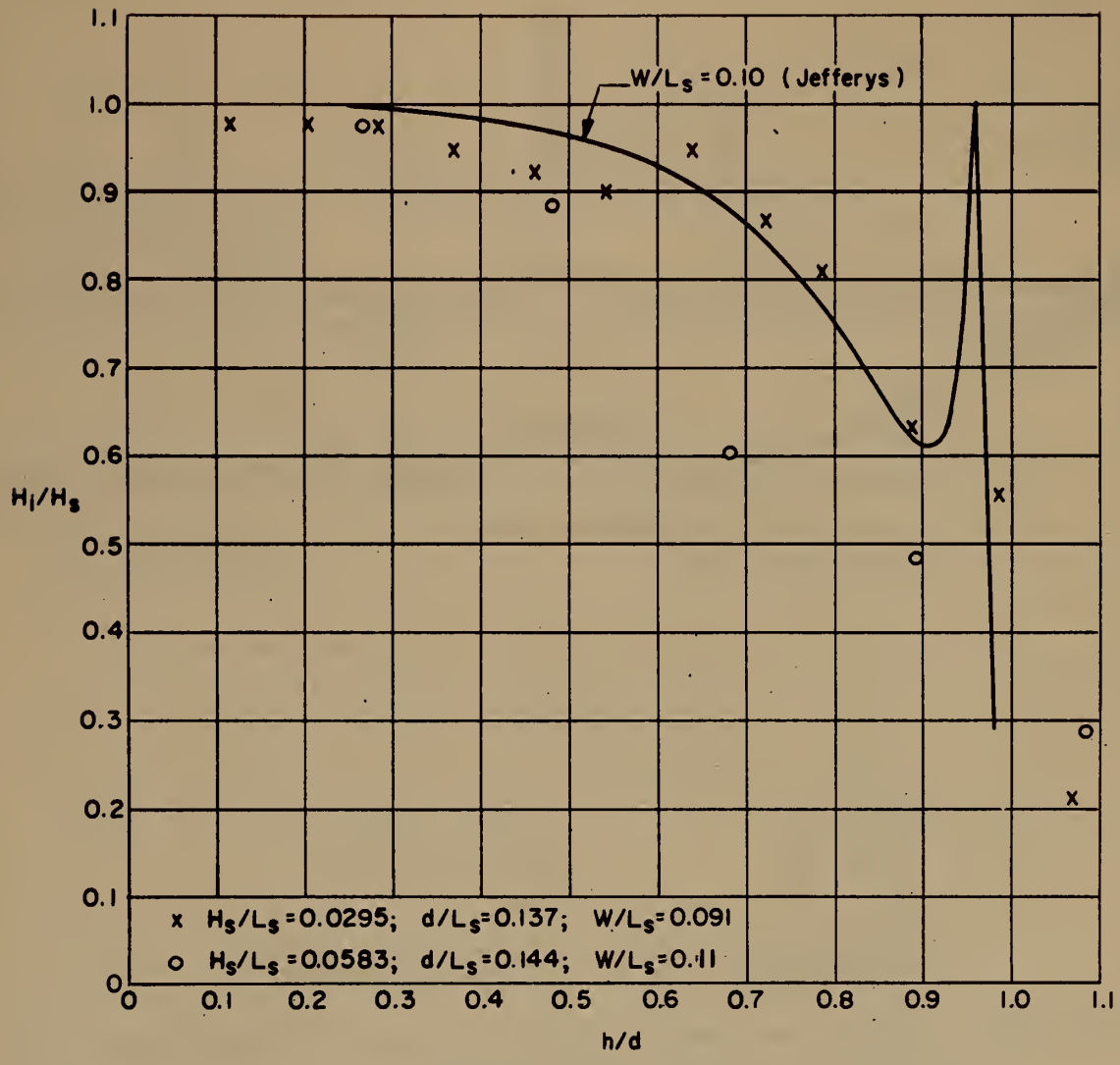



FIG. IXF-22 - Effect of wave steepness on wave action over a rectangular underwater barrier (48)

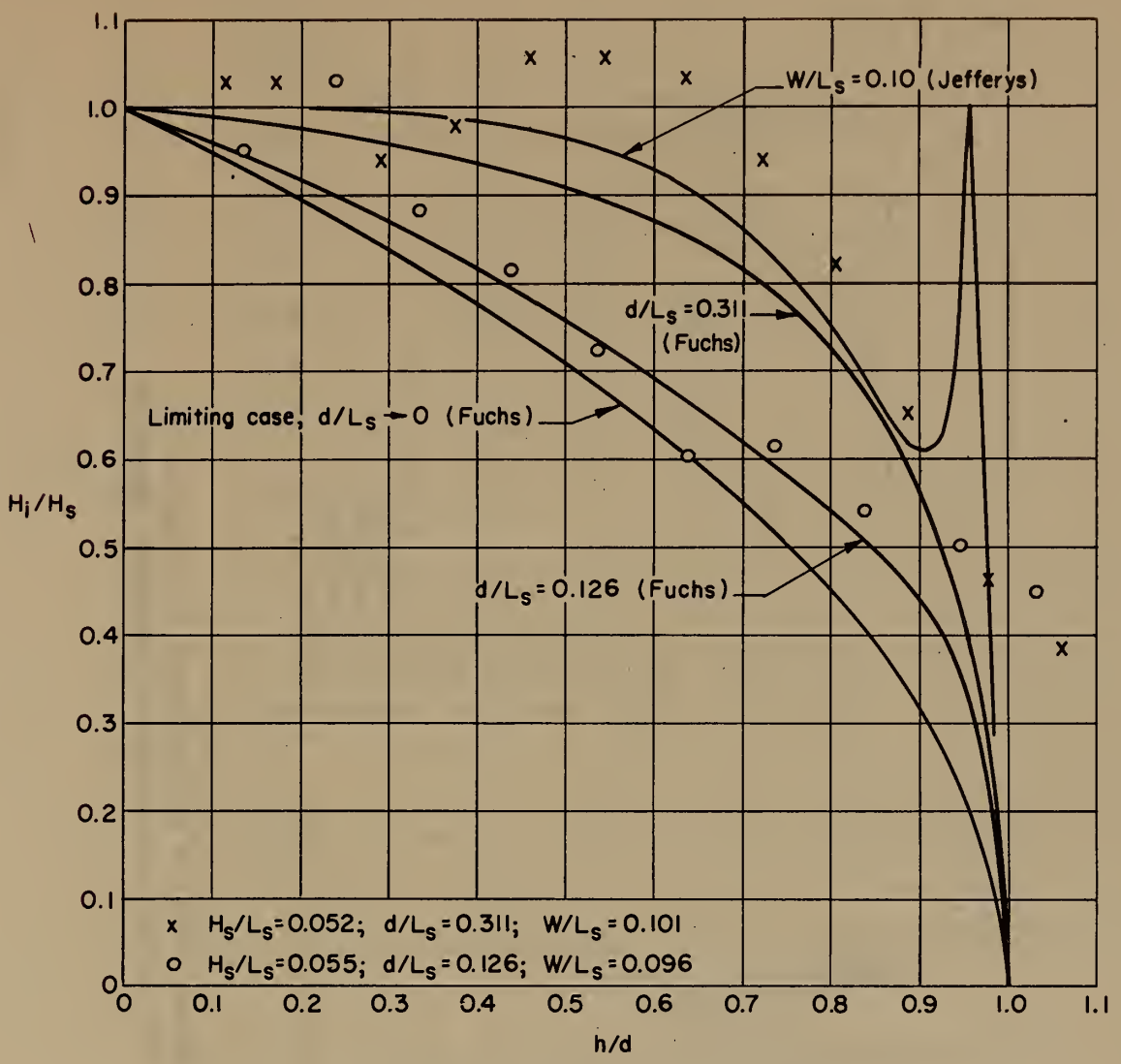
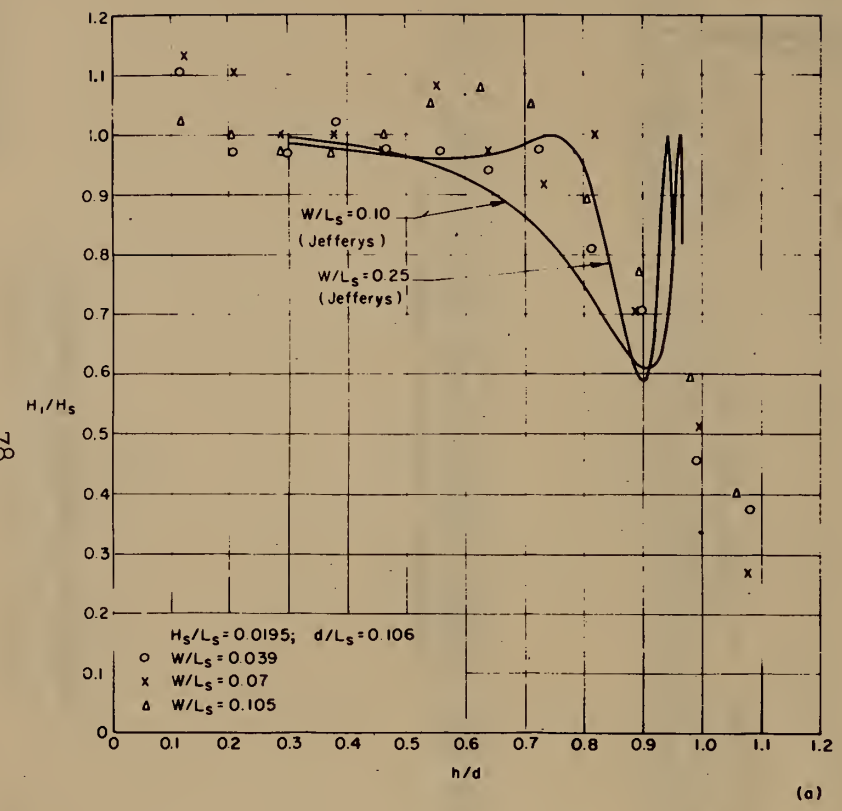



FIG. IX F-23 - Effect of relative depth on wave action over a rectangular underwater barrier (48)



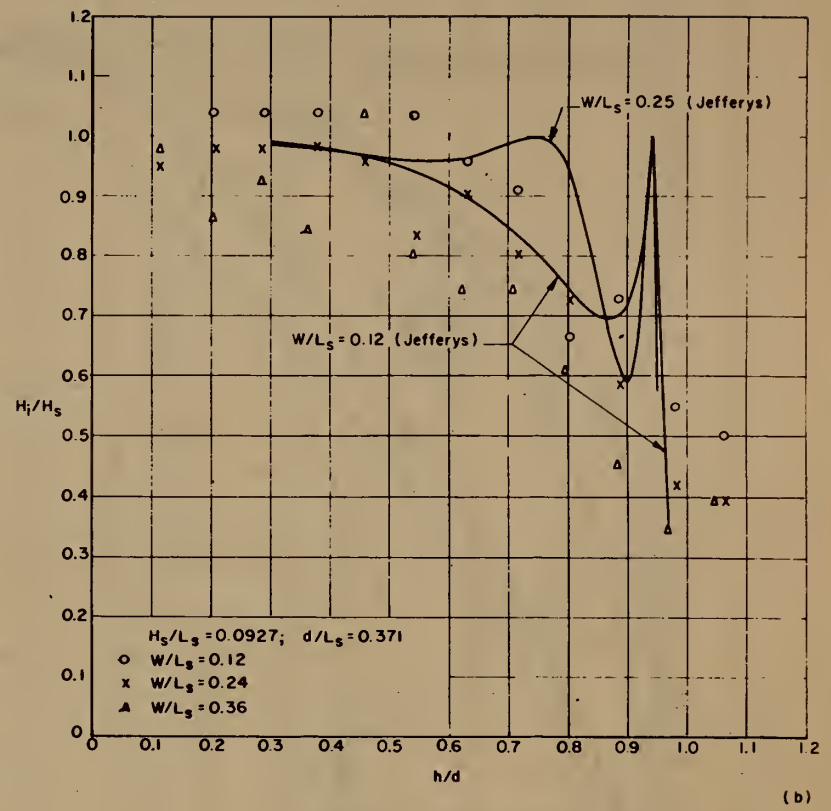




FIG. IXF-24 - EFFECT OF RELATIVE WIDTH OF A STRUCTURE ON WAVE ACTION OVER A RECTANGULAR UNDERWATER BARRIER (48)

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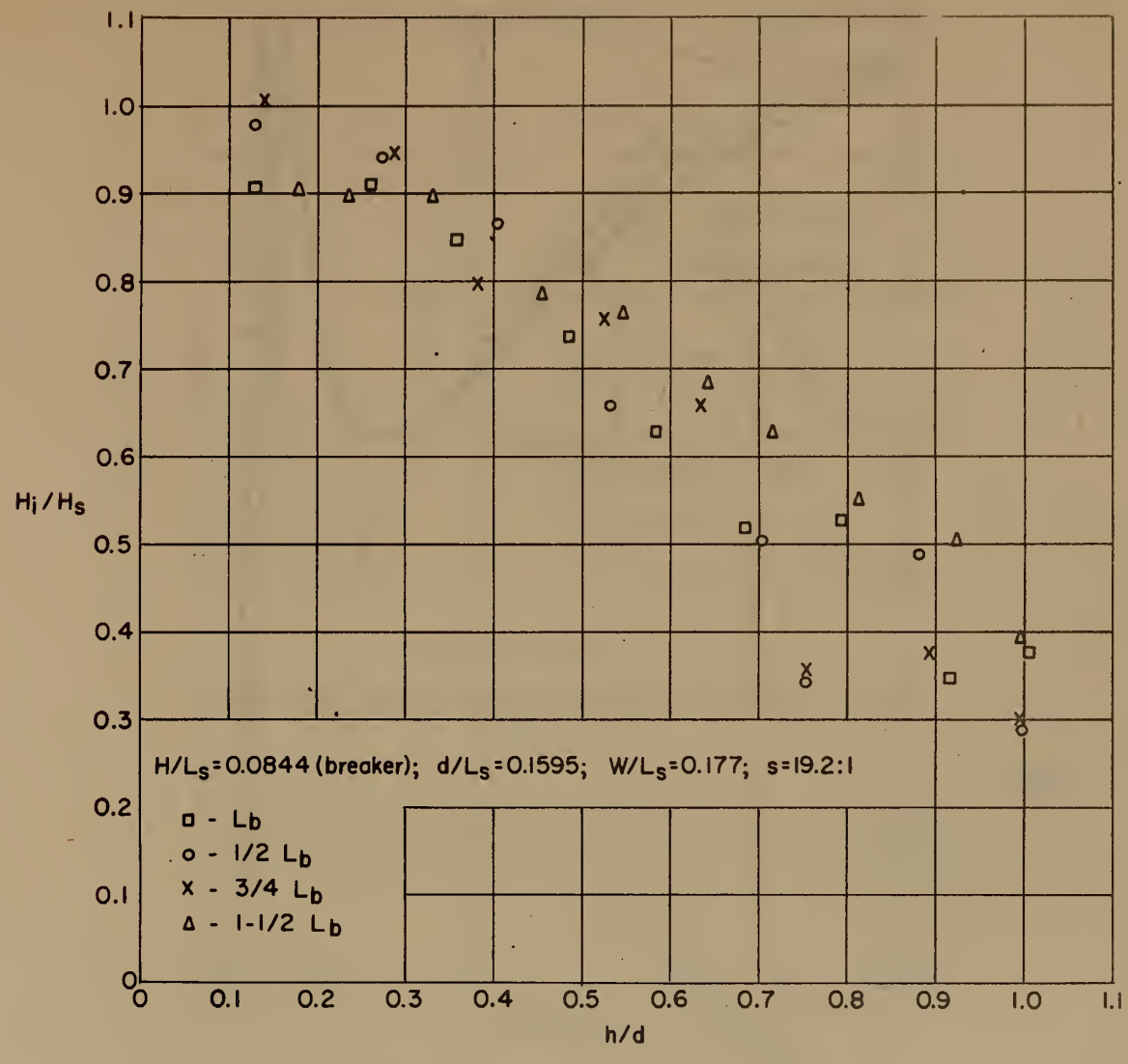



FIG. IX F-25 - Effect of barrier position on wave action over a rectangular underwater barrier (48)

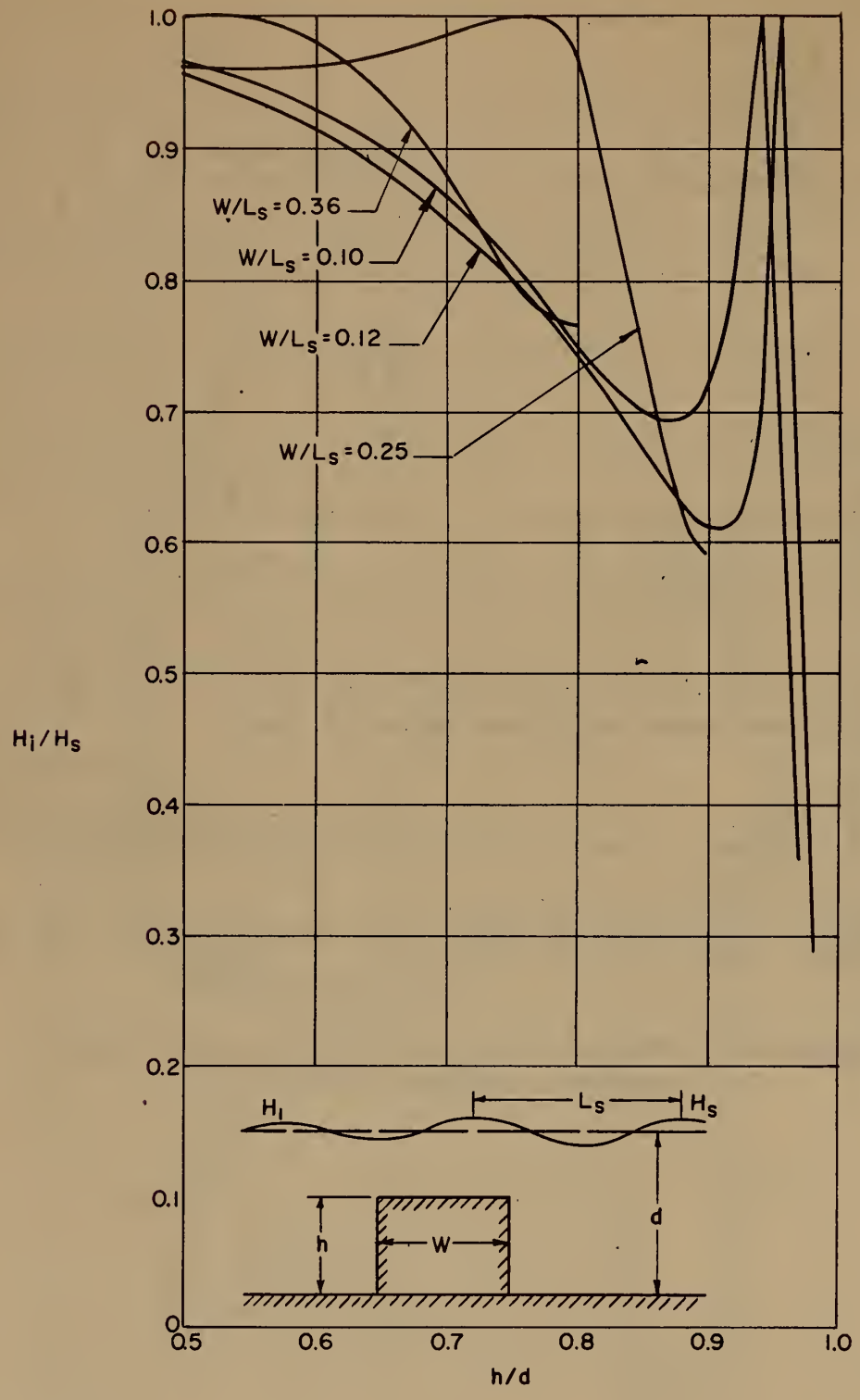



FIG. IX F-26 - Theoretical transmission coefficients obtained by Jefferys for wave action over a rectangular underwater barrier (48)

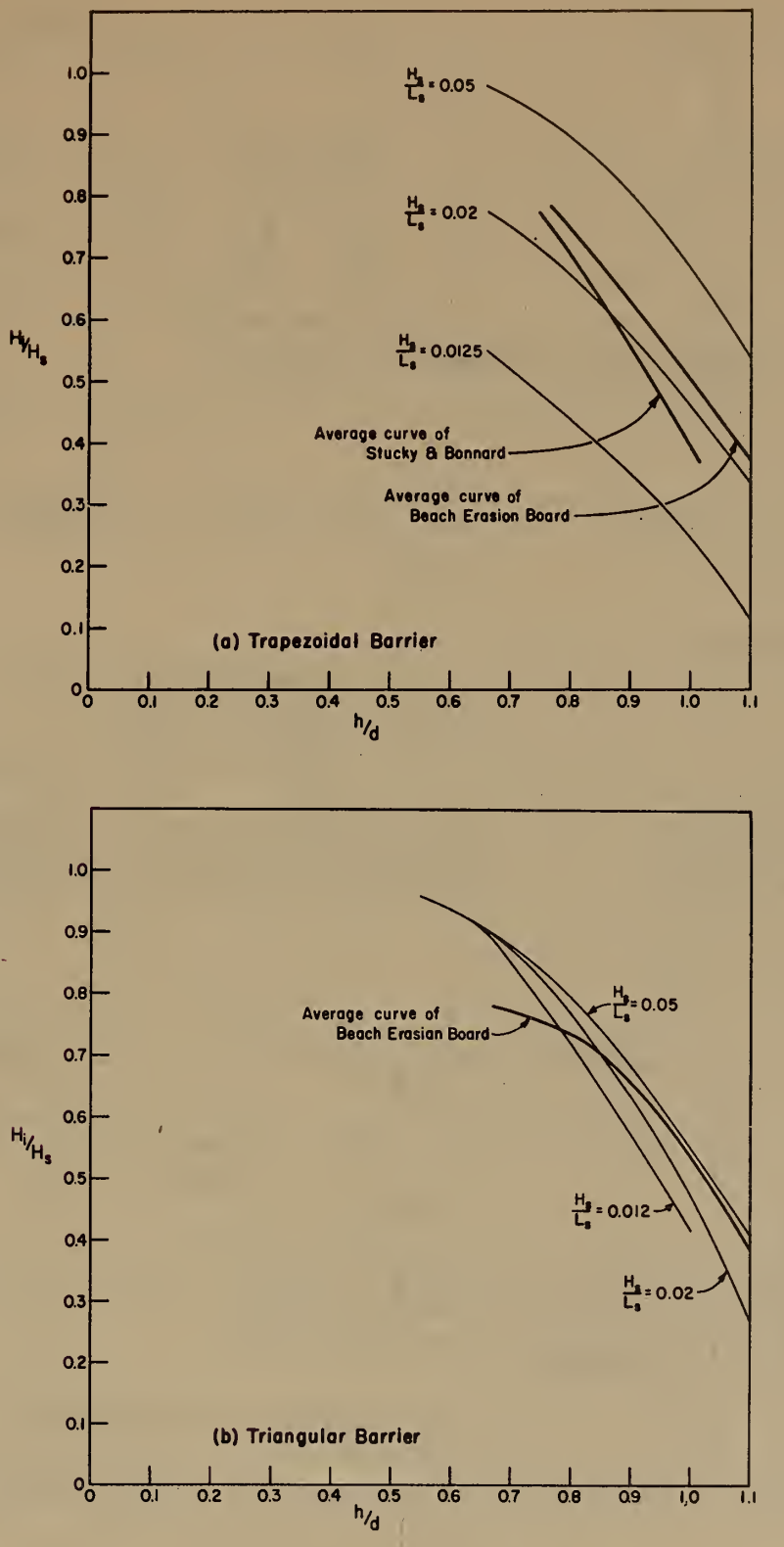



FIG. EXF-27 - Transmission coefficients for trapezoidal and triangular underwater barriers (from experiments by the Beach Erosion Board) (48)

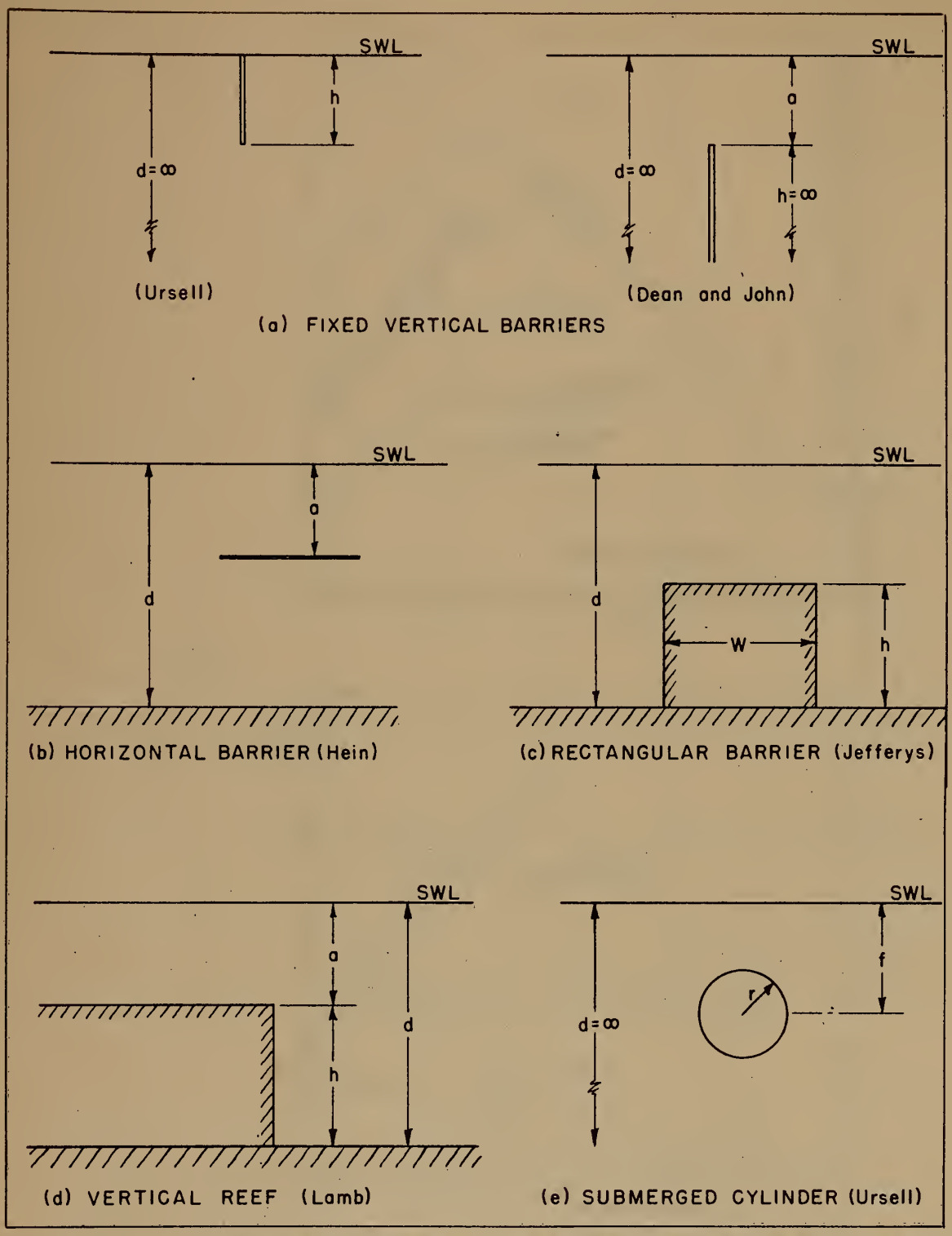



FIG. IXF-28 - Various types of underwater barriers investigated from theoretical considerations[48]

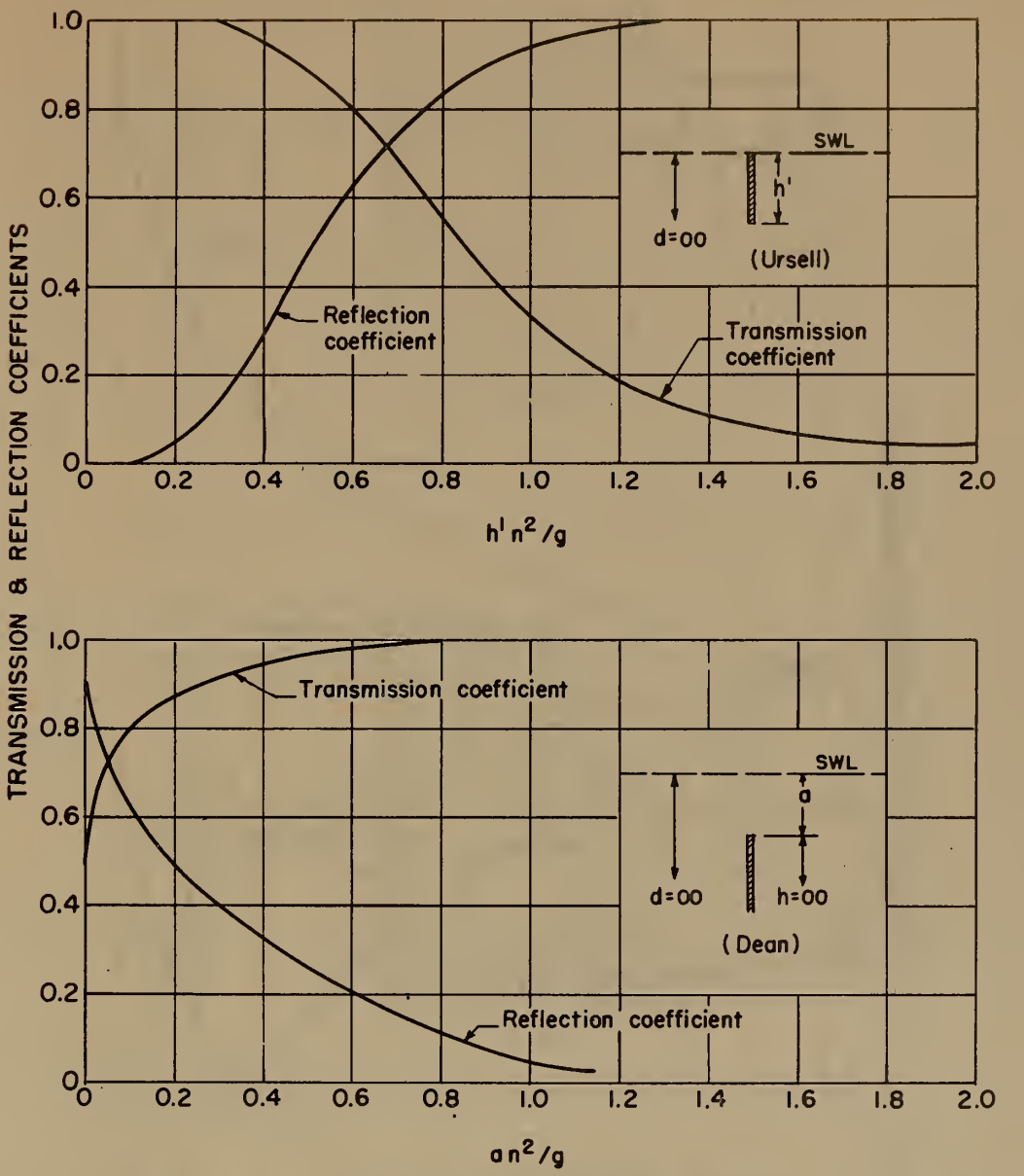



FIG. IXF-29 - Theoretical reflection and transmission coefficients obtained by Dean and Ursell for wave action at fixed vertical barriers (wave period = $2\pi/n$) (48)

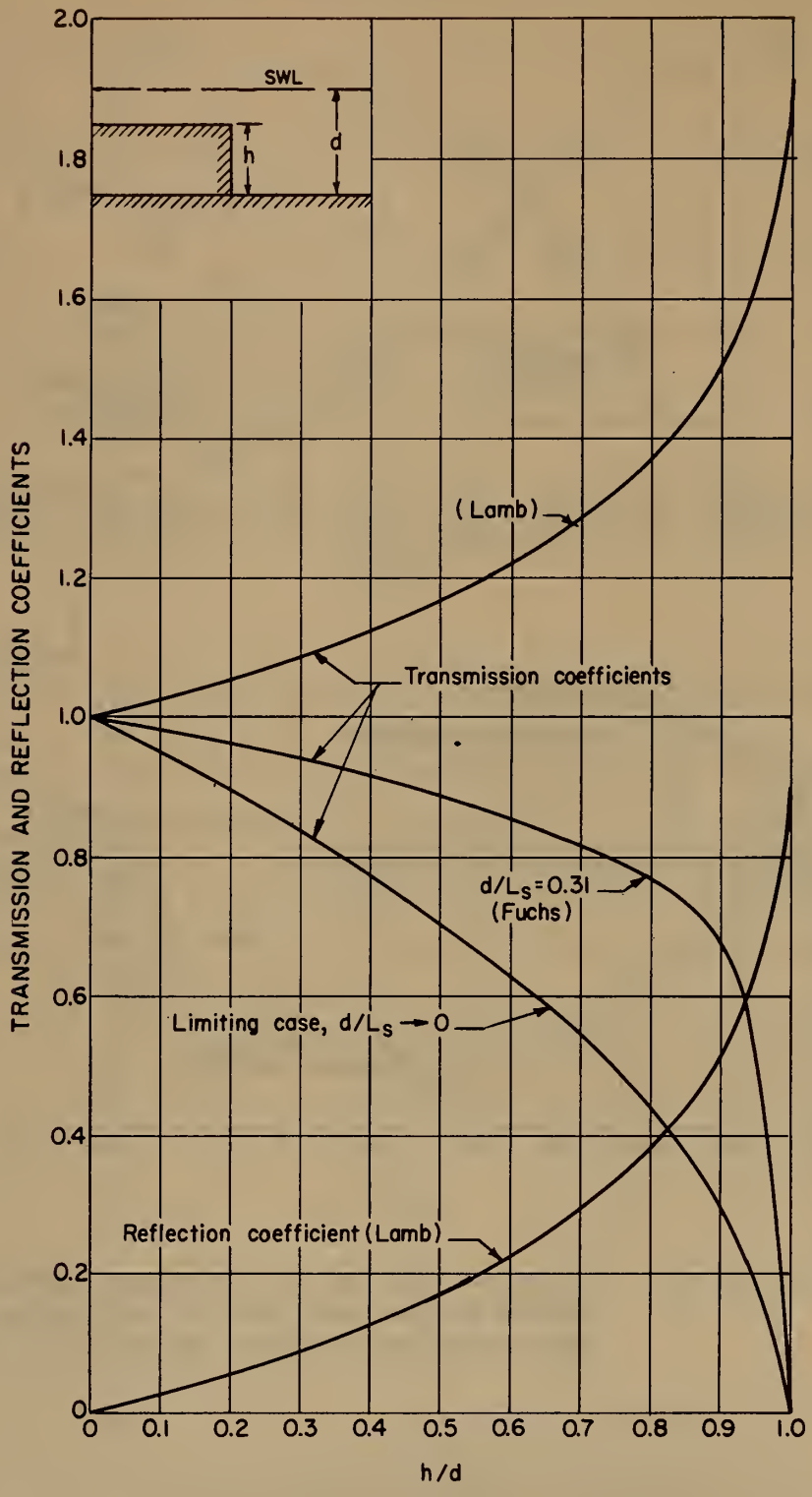



FIG. IX F-30 - Theoretical reflection and transmission coefficients for wave action on a vertical reef [48]

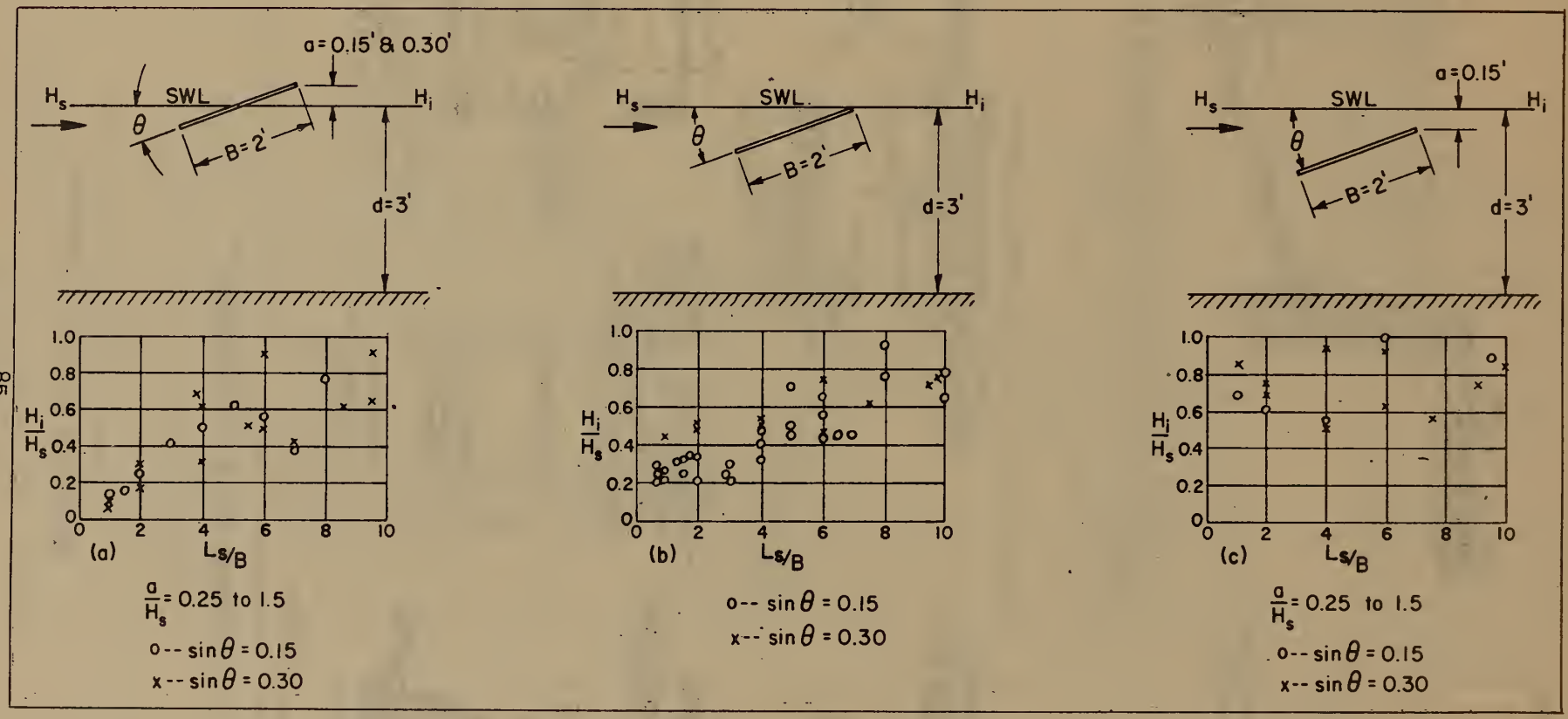
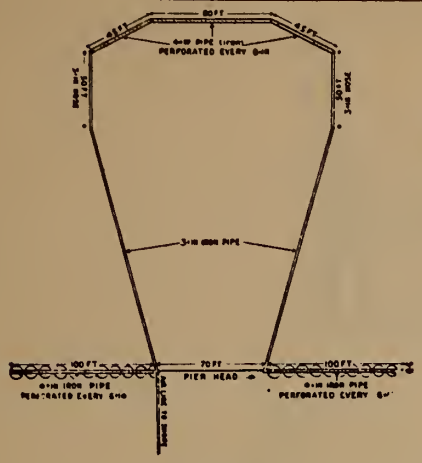
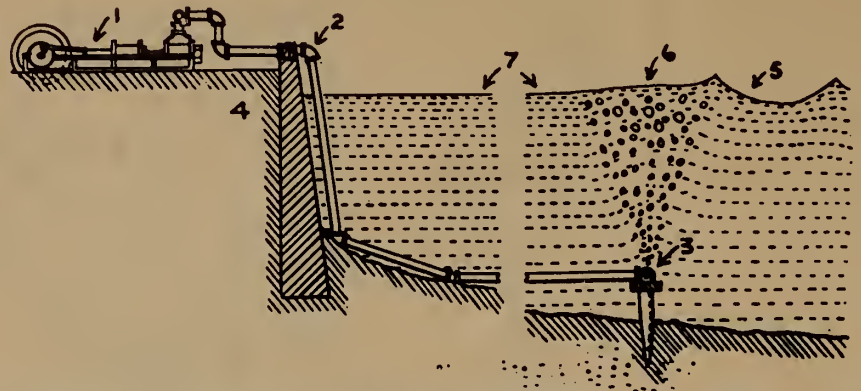
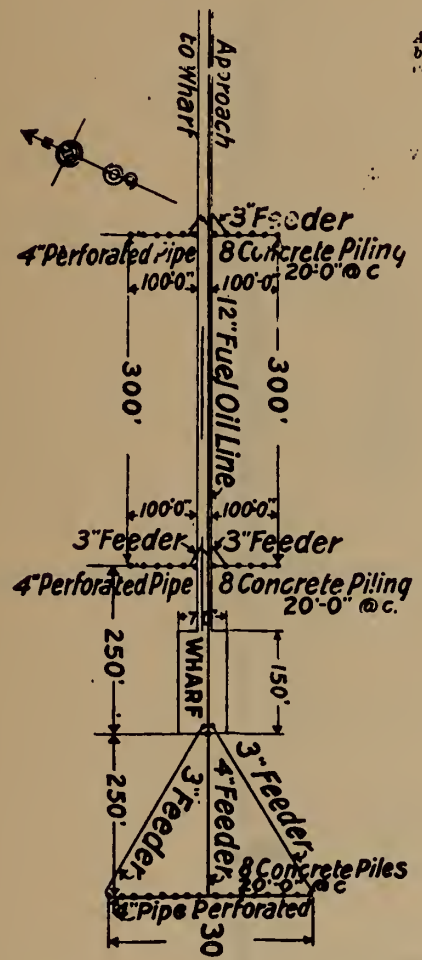



FIG. IX F-31 - Transmission coefficients for submerged fixed inclined barriers (by Hamilton) [48]

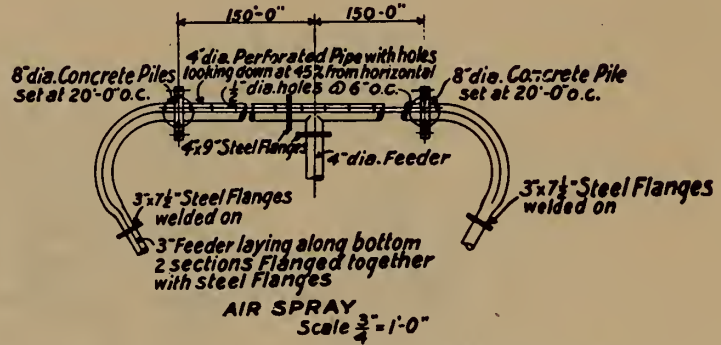


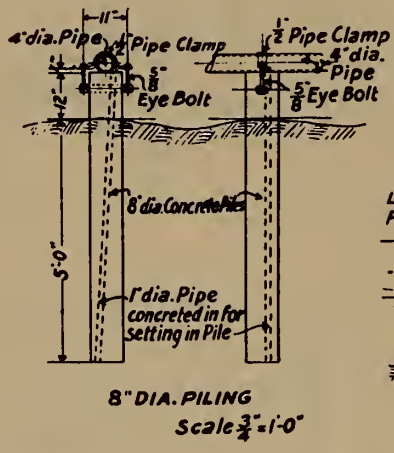



*A schematic sketch of the operative features of a Brasher compressed air breakwater. 1—Air compressor; 2—Air supply line to breakwater; 3—Perforated pipe of breakwater; 4—Pier to be protected; 5—Area of rough water; 6—Ascending and expanding air bubbles. 7—Area of calmed water.*




*Upper, 1st installation; lower, 2d installation at El Segundo.*



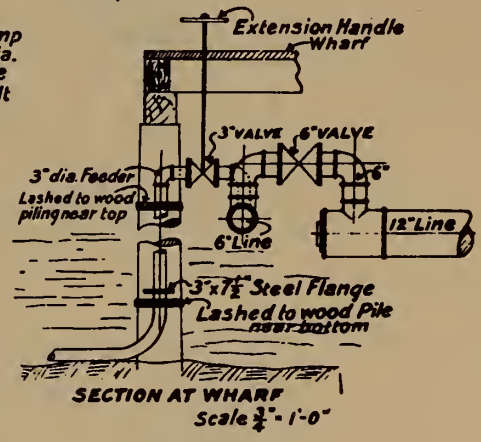
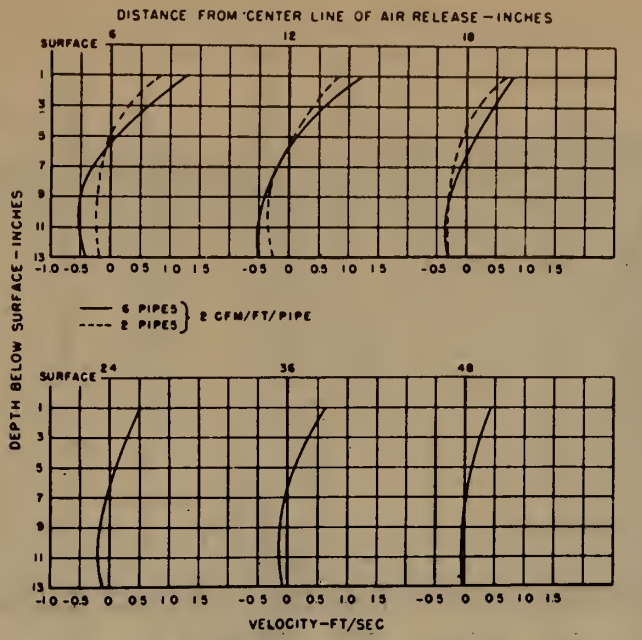
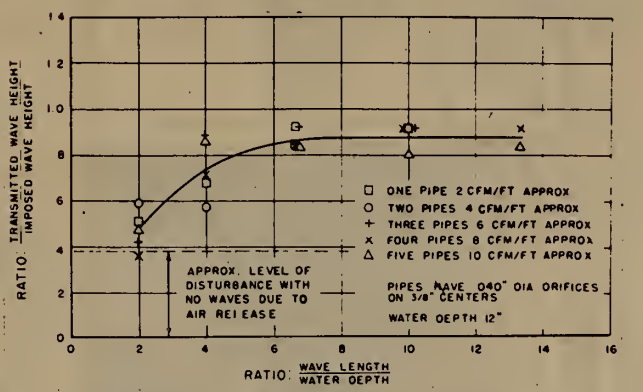



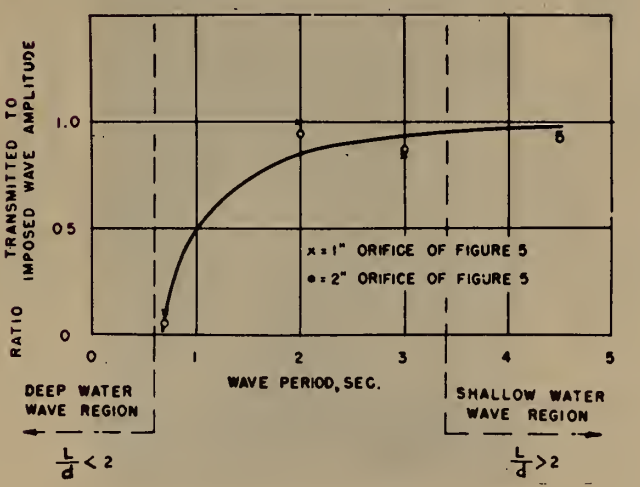

FIG. IXF-32 - Air breakwater set-up at El Segundo (74)




a. Water currents produced by a model pneumatic breakwater




b. Pneumatic breakwater performance




c. Performance of water jet breakwater

FIG. IX F-33 - Model studies of air and water set breakwaters (13)

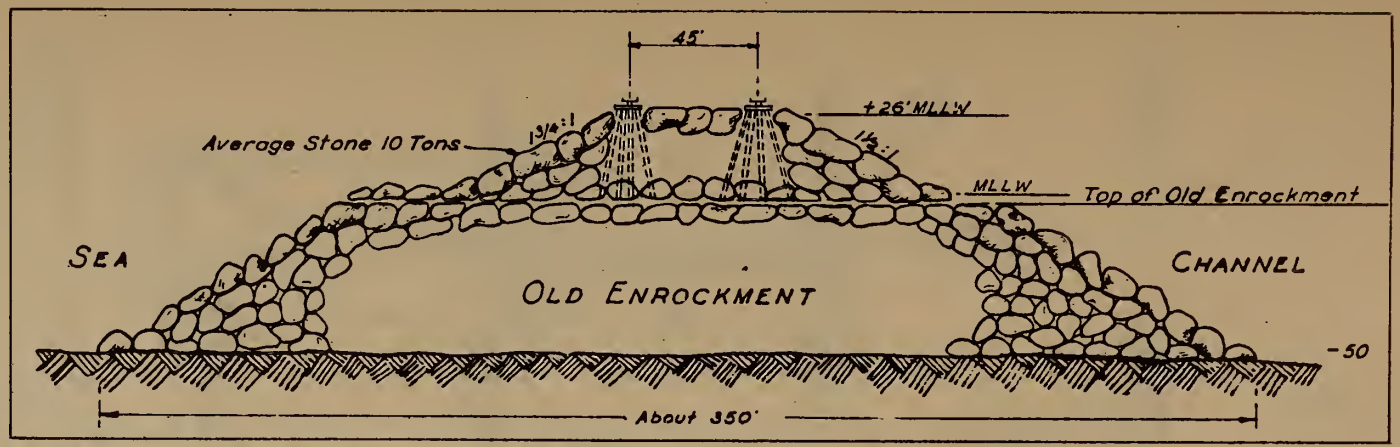
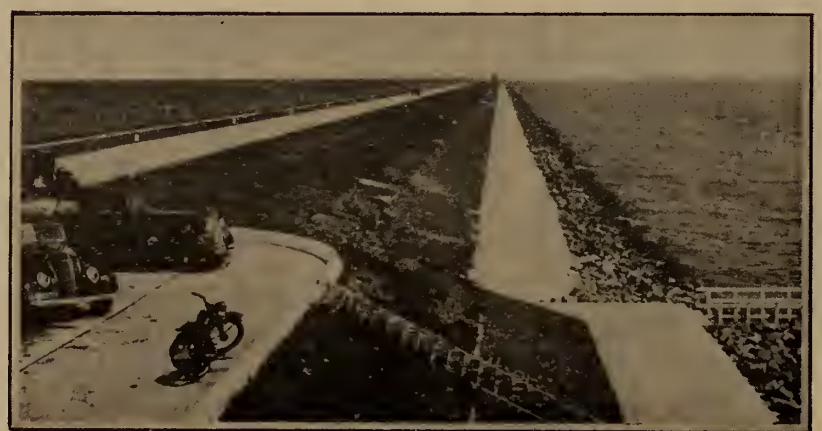



FIG. IXF-34 - Typical section of south jetty, mouth of Columbia River, Oregon, as reconstructed in 1935 of random stone (23)



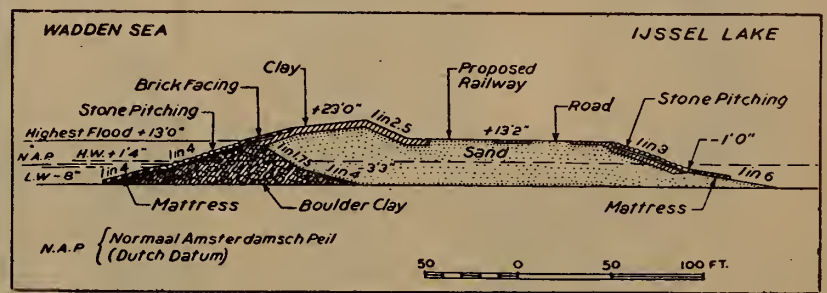



FIG. IXF-35 - Cross-section of the Zuyder Zee Dyke (63)

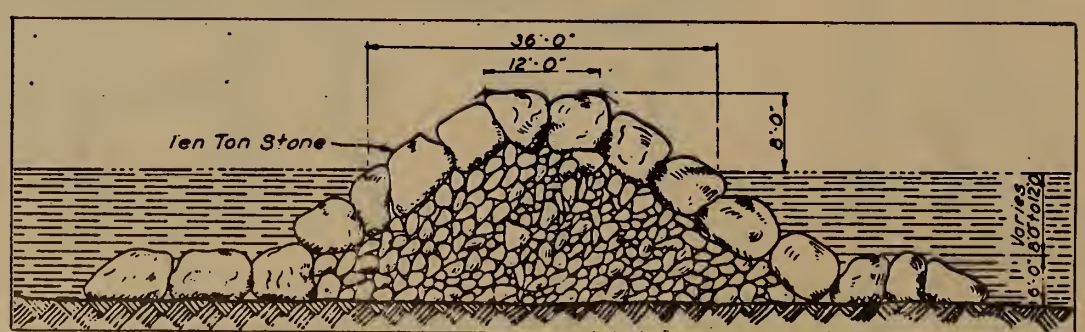



FIG. IXF-36 - Rubble breakwater at Presque Isle Harbor, Michigan, on Lake Superior; random stone with block facing (23)



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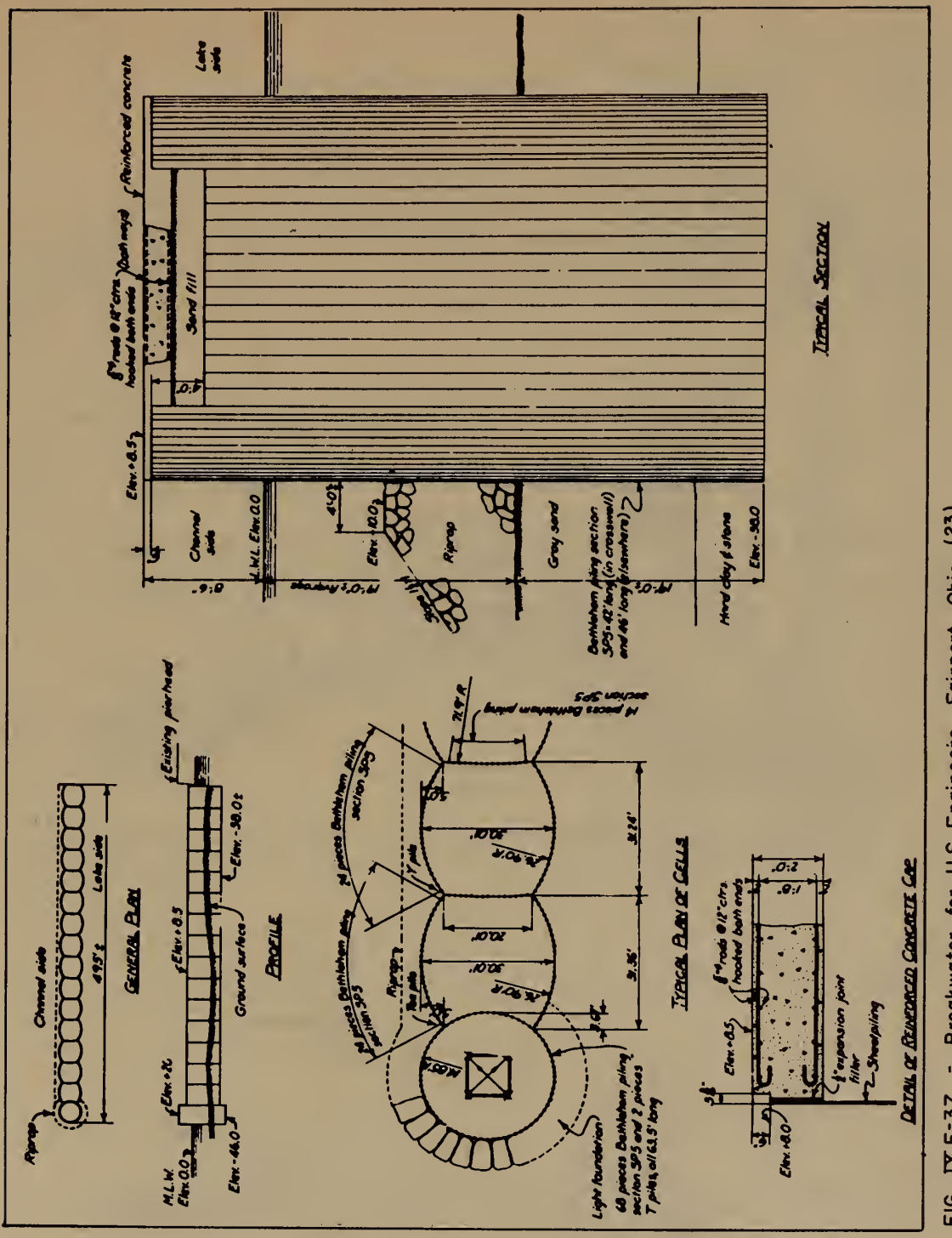



FIG. IX F-37 - Breakwater for U.S. Engineers, Fairport, Ohio (23)

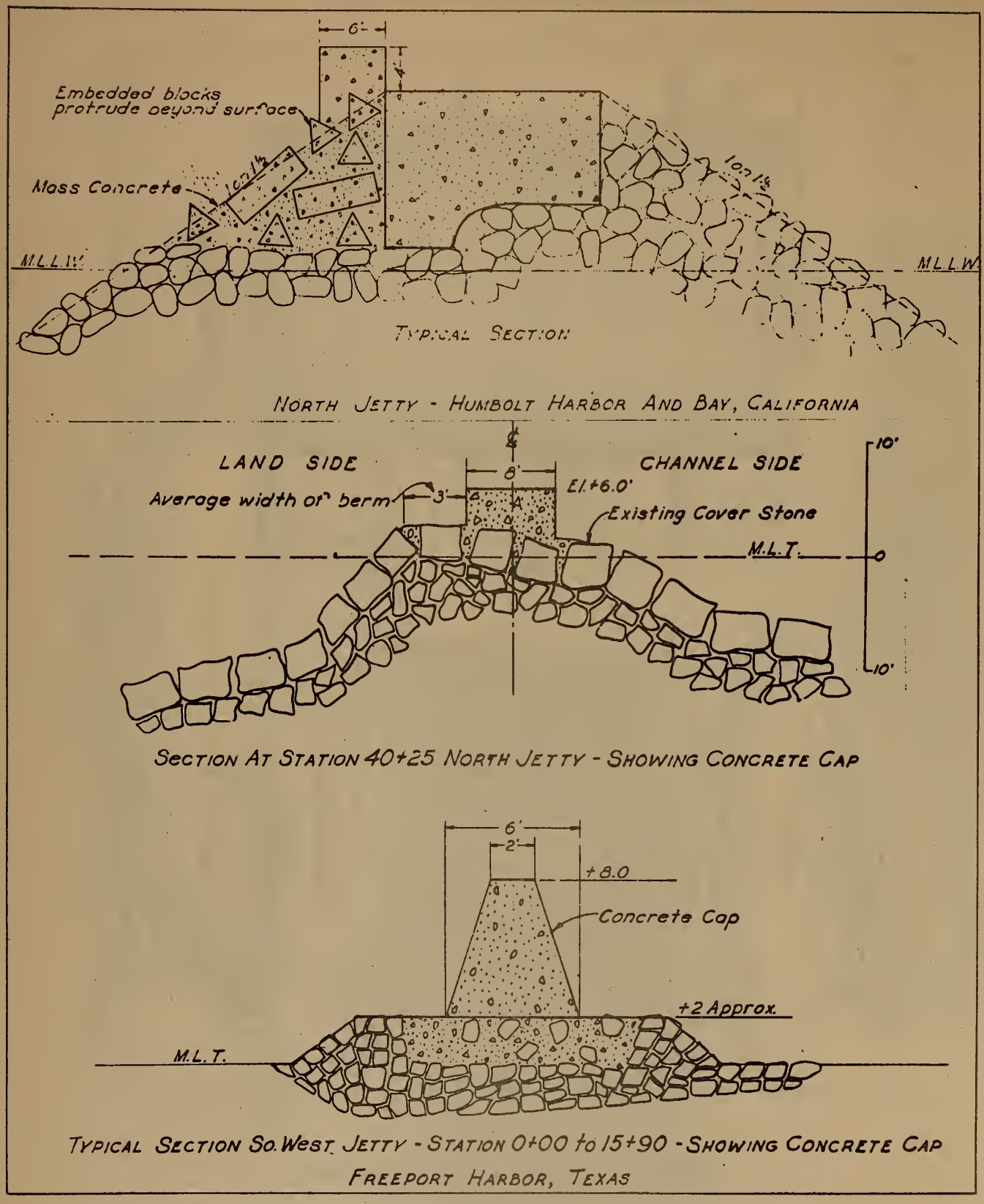



FIG. IX F-38 - Concrete caps on rubble mounds [23]

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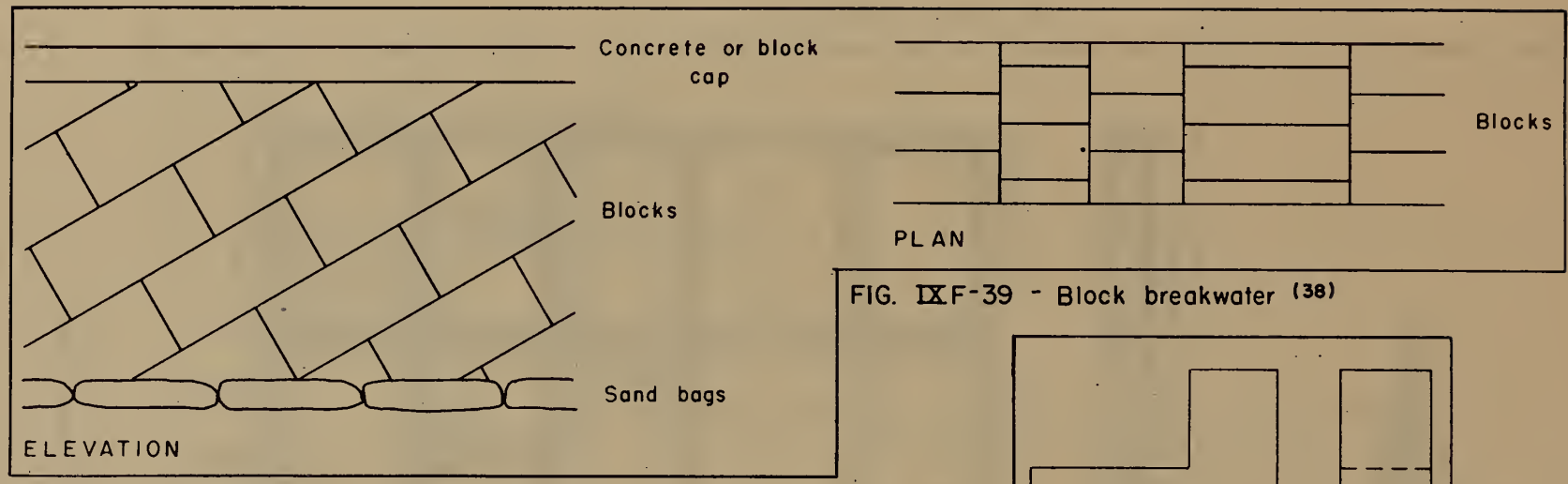
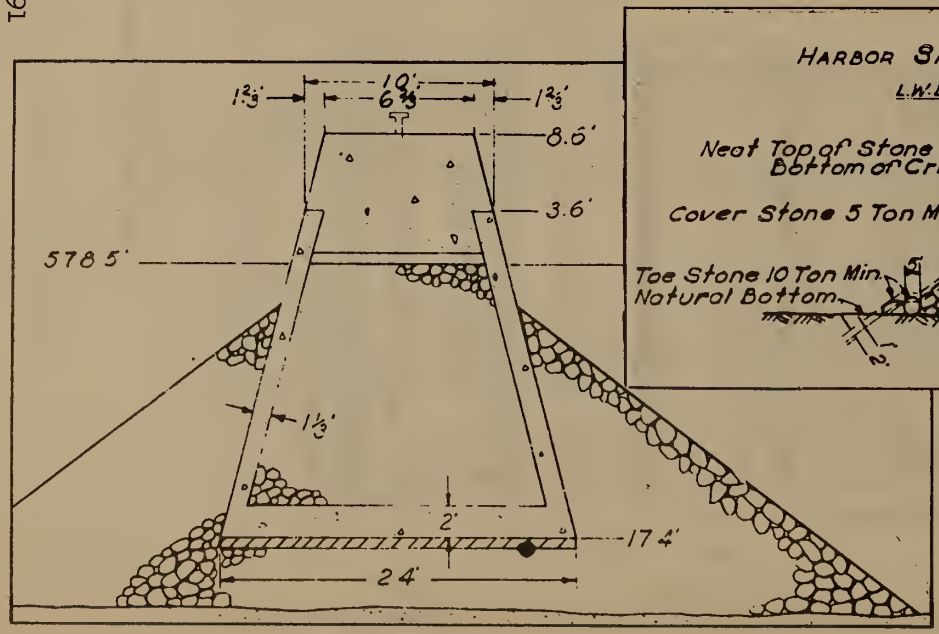



FIG. IXF-39 - Block breakwater (38)



FIG. IXF-40 - Piano block (38)



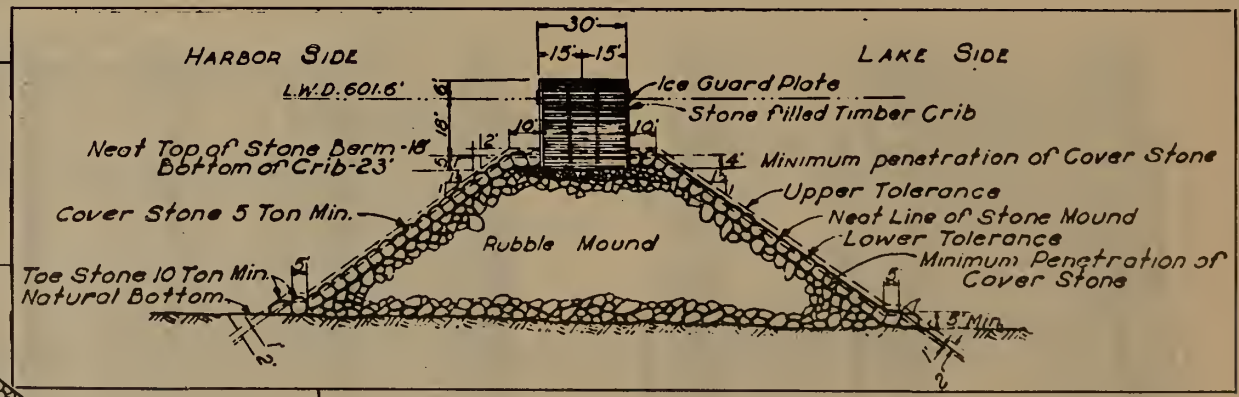




FIG. IXF-42 (above) - Breakwater section, Two Harbors, Minnesota (23)

FIG. IXF-41 - Typical section, north and south breakwaters, Milwaukee harbor, Wisconsin (23)

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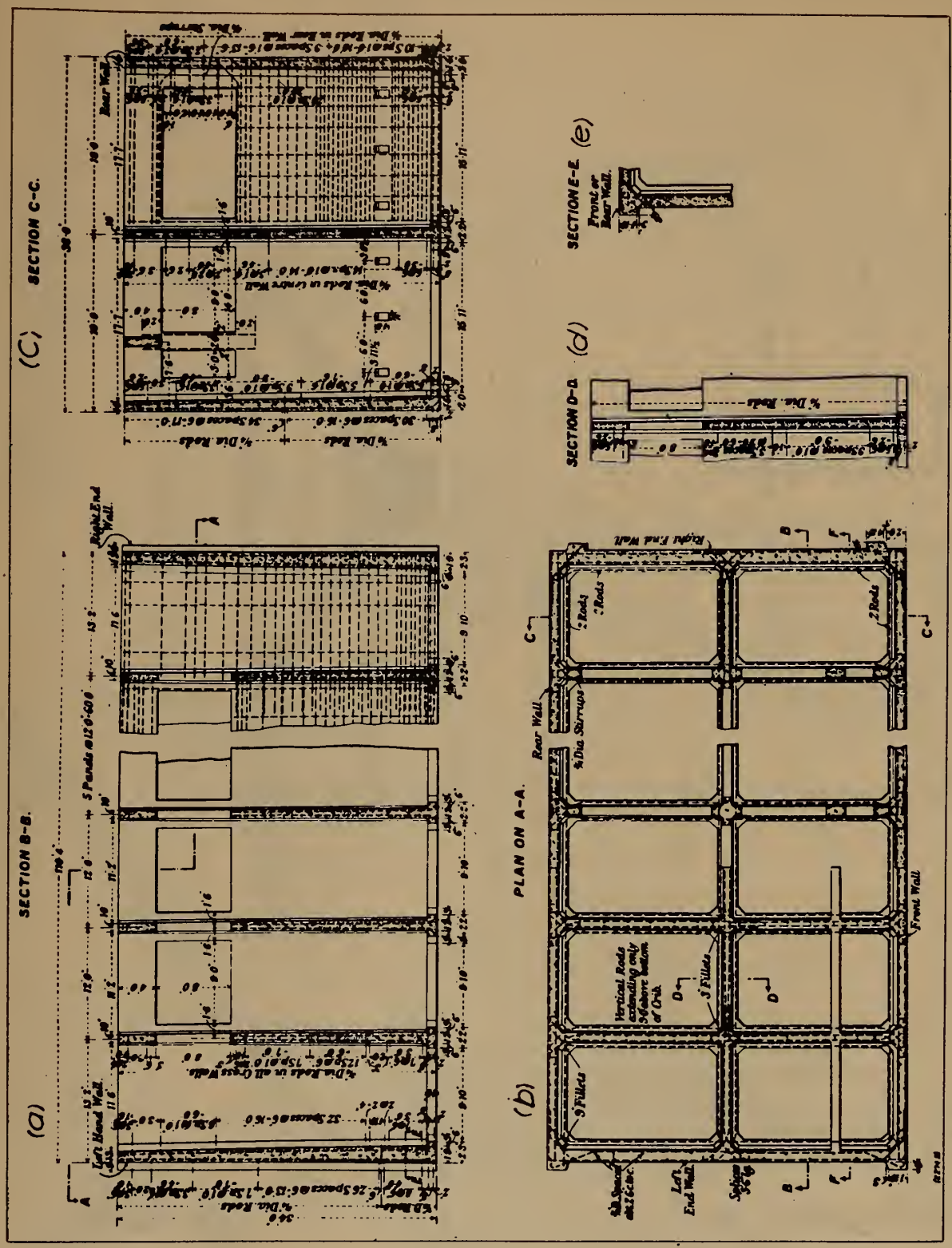



FIG. IX F-43 - Concrete caissons without permanent bottom used on the Welland Ship Canal by the Department of Railways and Canals, Canada (23)

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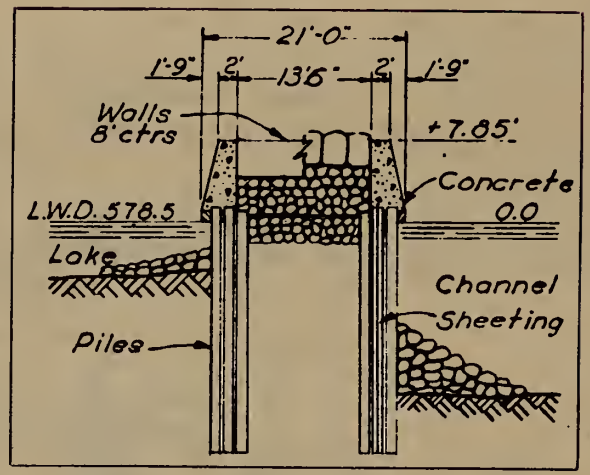



FIG. IXF-44 - Typical section of west pier, Michigan City harbor, Indiana (23)

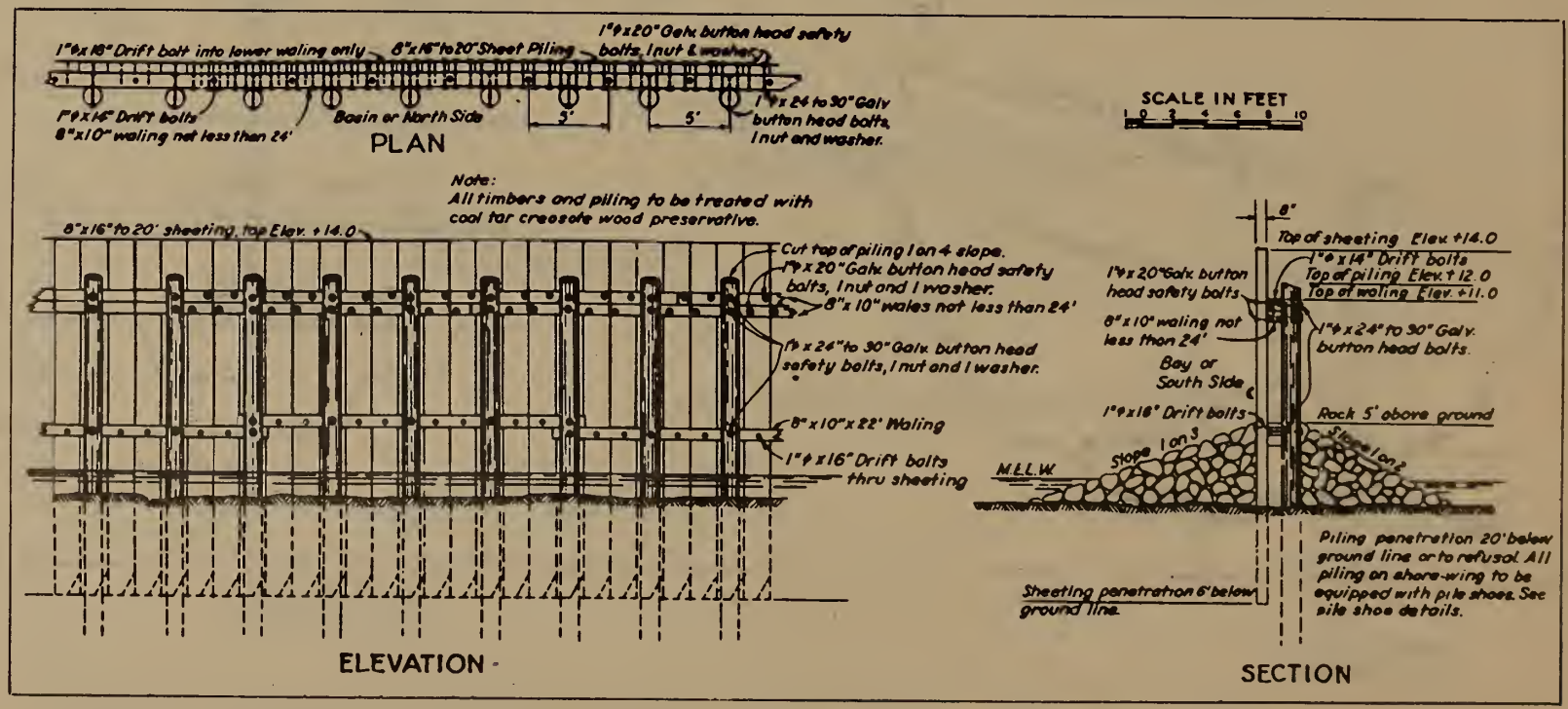



FIG. IXF-45 - Details of timber breakwater for small boat mooring basin, Yaquina Bay, Washington (23)

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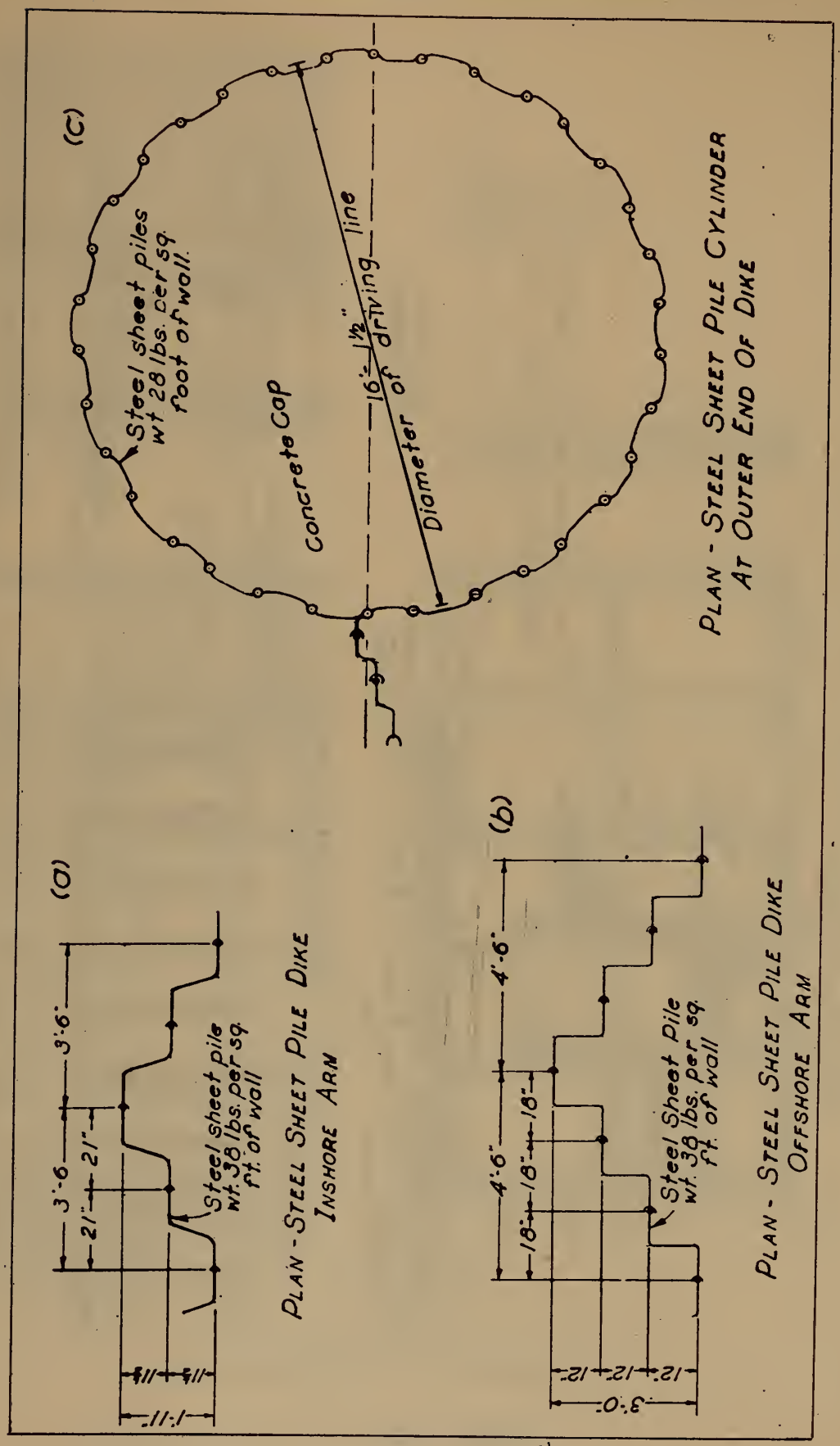



FIG. IX F-46 - Port O'Connor dikes, Port O'Connor, Texas (23)

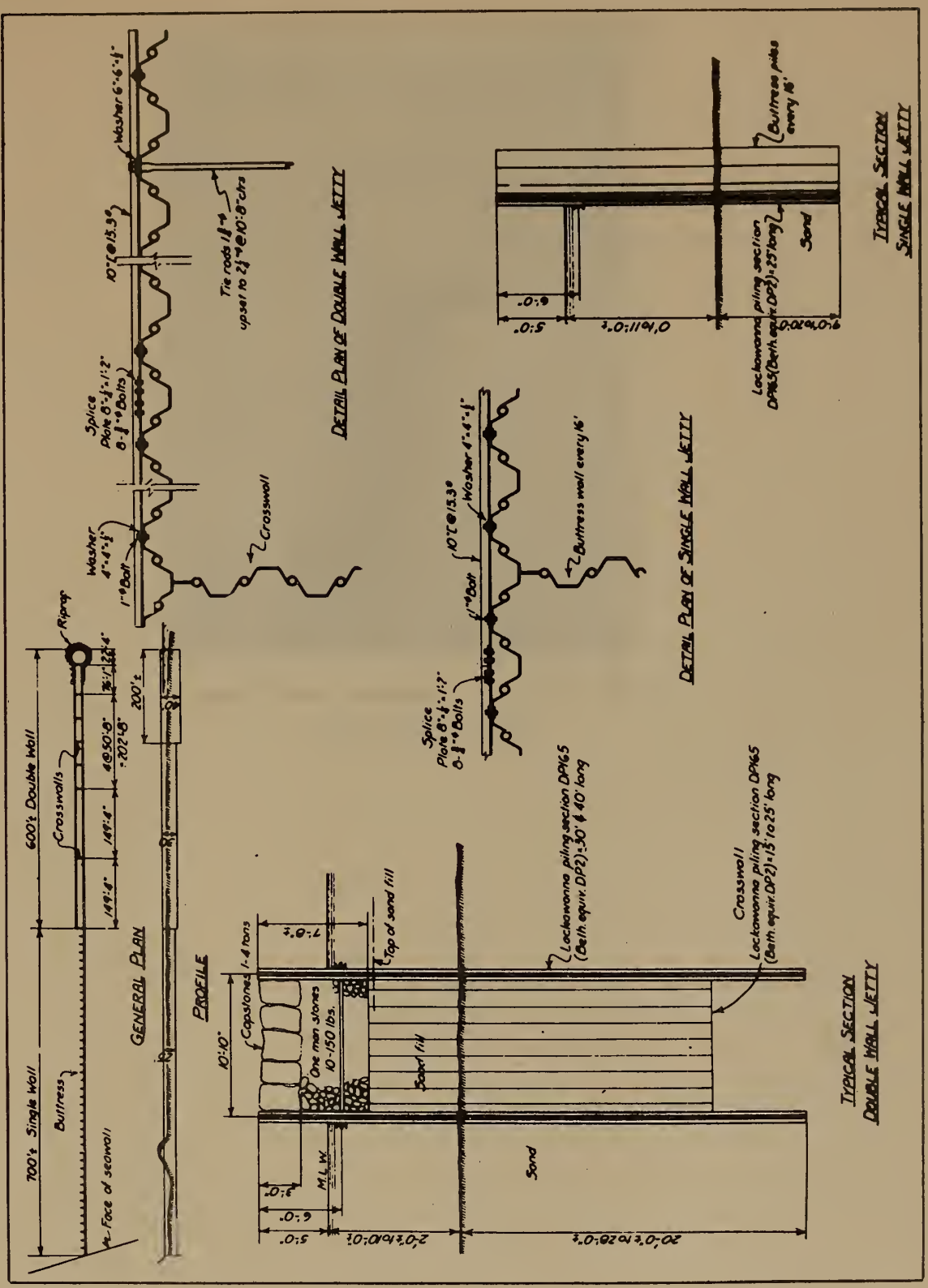



FIG. IX F-47 - Sheet-pile jetty for beach protection at Fort McRee, Pensacola Bay, Florida (23)



FIG. IX F-48 - Seaplane basin breakwater, Midway (20)

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G. JETTIES

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## G. JETTIES

The purpose of a jetty is to control and direct the outflow of rivers and harbor entrances and protect these from wave action. They are built with the idea of causing the outflow current to be rapid enough to scour a navigable channel or at least enough of a channel so that the river does not back up and meander through a series of lagoons and marsh lands.

The construction of a jetty is similar to breakwater construction. The physical considerations of a jetty constructed where it is subject to severe wave action are the same as for breakwaters. In many cases, the reference to jetties and breakwaters is synonomous.

The direction of jetties used for harbor entrances should be in the direction of the most frequent severe wave action so that ships may enter with the waves. This also minimizes the depositing of sand in the entrance during the heaviest storms. The channel opening of a jetty system used for navigation must be wide, at least 200 feet for large ships.

# SECTION IX: INVASION HARBORS

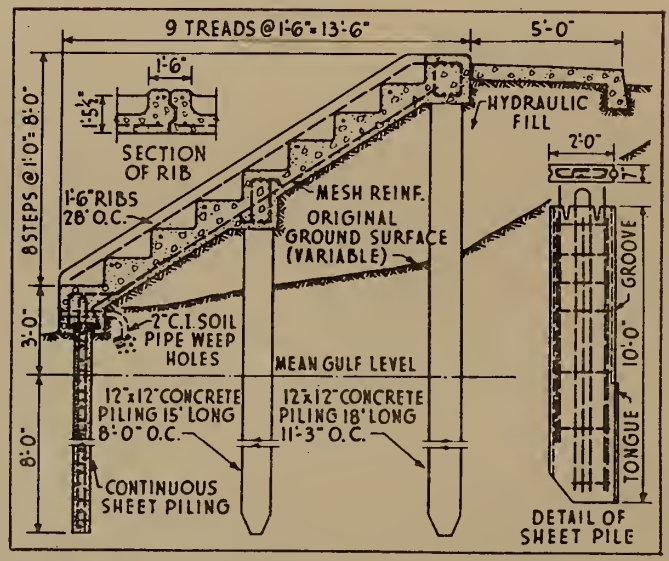
## H. SEAWALLS

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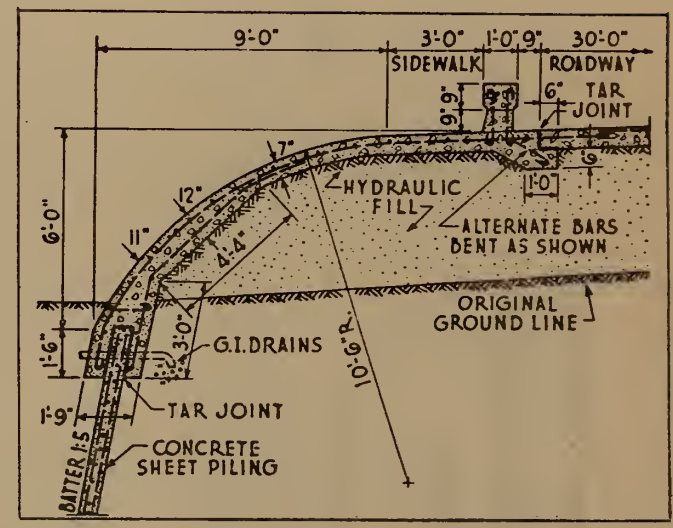
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## H. SEAWALLS

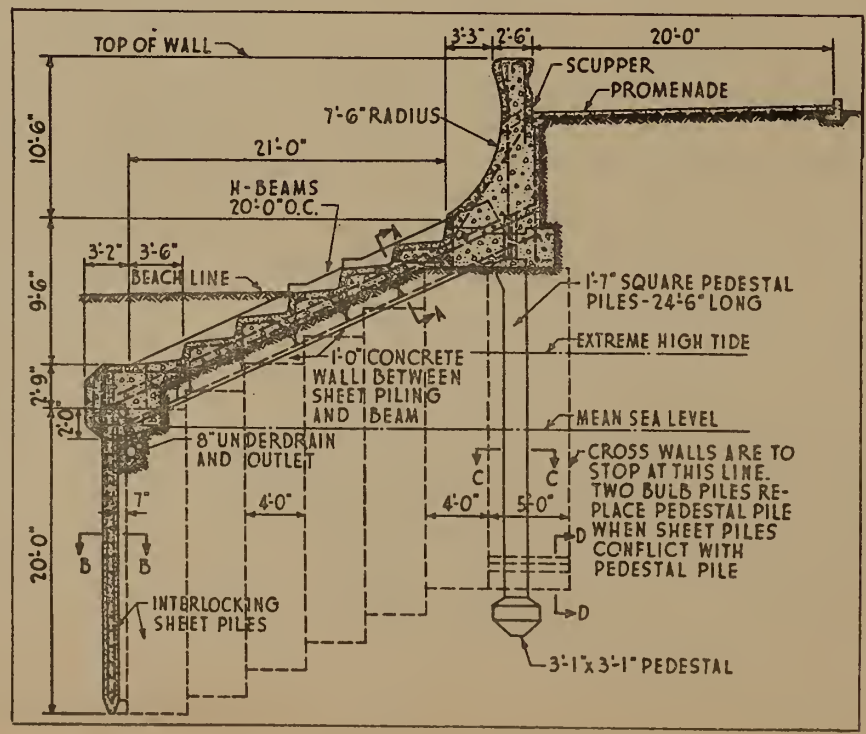
The protection of the shoreline proper and inner harbor shore is the function of a seawall. The resistance to wave action is mainly through the wall and backfill. Figure IX H-1 illustrates some types of concrete seawalls that have been built. Seawall construction may be similar to breakwaters and jetties. The erosion of the toe of a seawall must be avoided by driving sheet piling and placing a rip-rap. The backfill material must be drained and confined by cut off wall and panel surfaces. Whenever a seawall is built it will change the formation of the beach. The size of a seawall depends on the size of waves expected to break against it, and on the tides. The changes in the beach after the seawall is built may affect the required size of the wall. Bulkhead type seawalls are used for wharves in a harbor where the water is relatively deep.



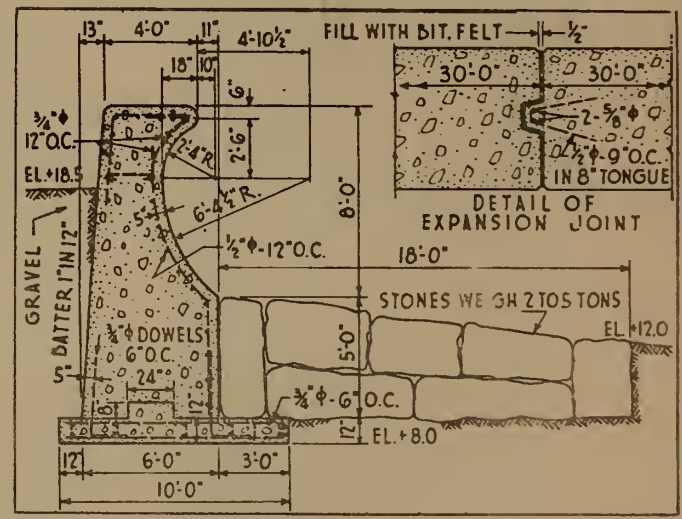

a. Sea wall, Harrison County, Mississippi




b. Convex sea wall, Jackson County, Mississippi




c. San Francisco esplanade sea wall




d. Sea wall at Hampton Beach, New Hampshire

FIG. IX H-1 - Sea walls



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## SECTION IX: INVASION HARBORS

## I. GROINS

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# I. GROINS

The purpose of a groin is to reduce beach erosion or to stop it. A groin system can affect accretion only if material is being supplied to the beach. The material affected by groins is sand and gravel. There are generally two types of groins--impermeable and permeable. Figures IX I-1 to 4 are examples of groins. Groins should be spaced (at a distance from each other) from 1 to 3 times their length. The combination of seawalls and groins system seems to be effective if the seawall is built at the high-water level and the groins extend out slightly beyond the low-water level. A groin should be built at the height that the beach is desired and with the desired profile out to the low-water mark and then horizontal. Groin systems do not always accomplish their purpose and should be built with great care and caution.

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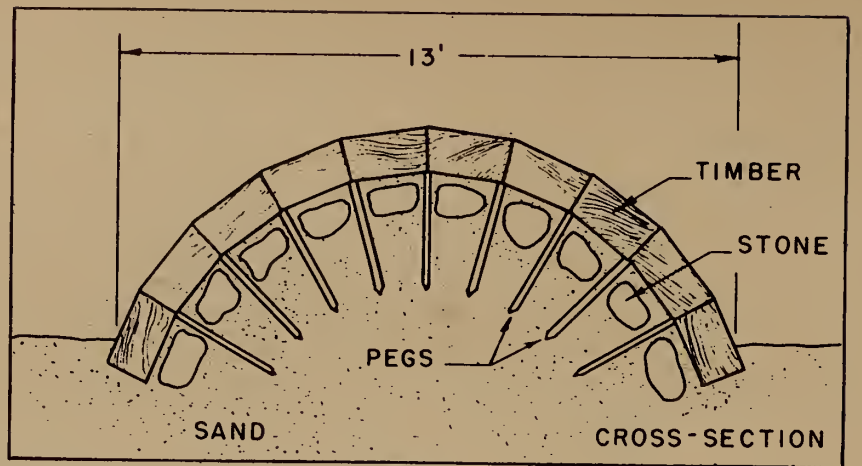



FIG. IX 1-1 - Simple groin (38)

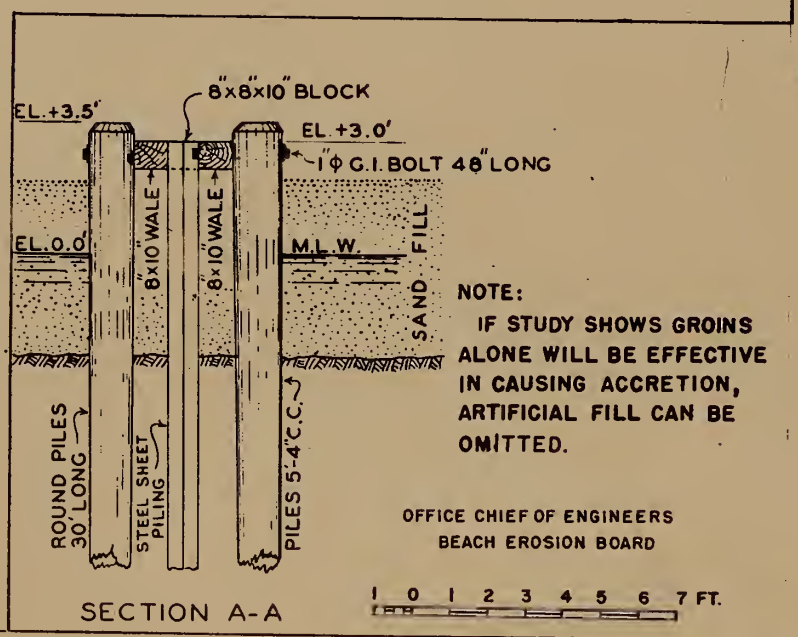
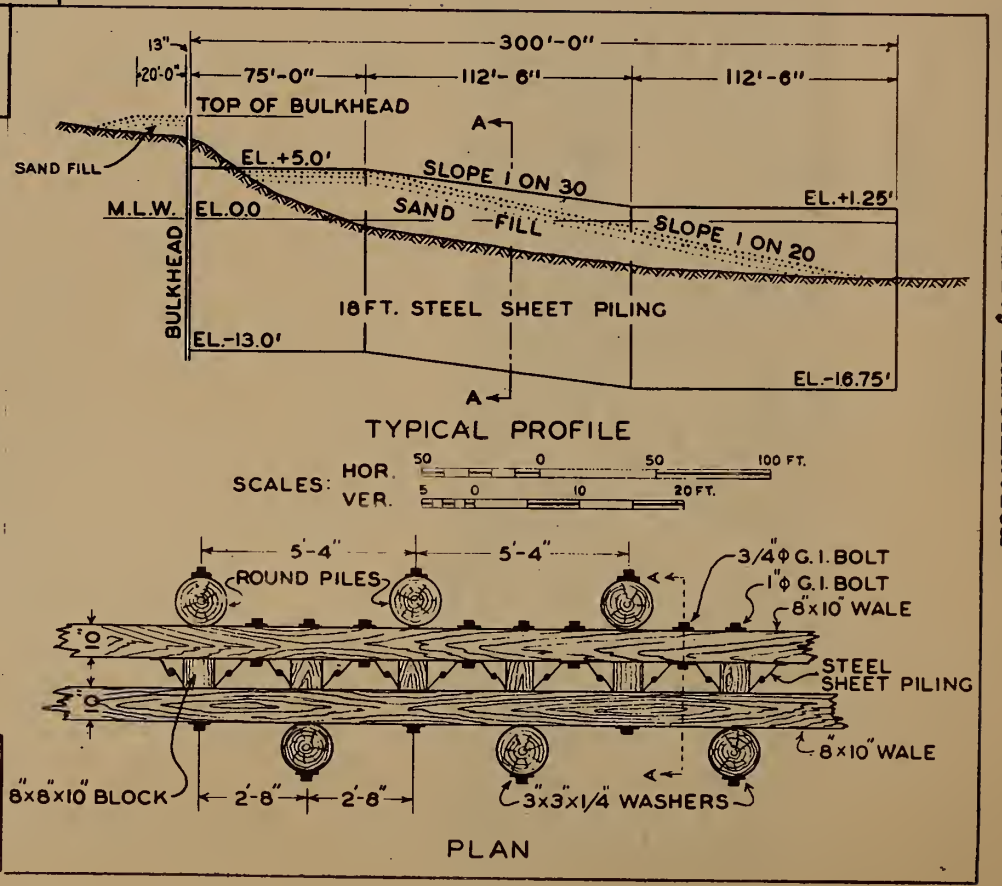



FIG. IX 1-2 - Typical groin with artificial fill (41)




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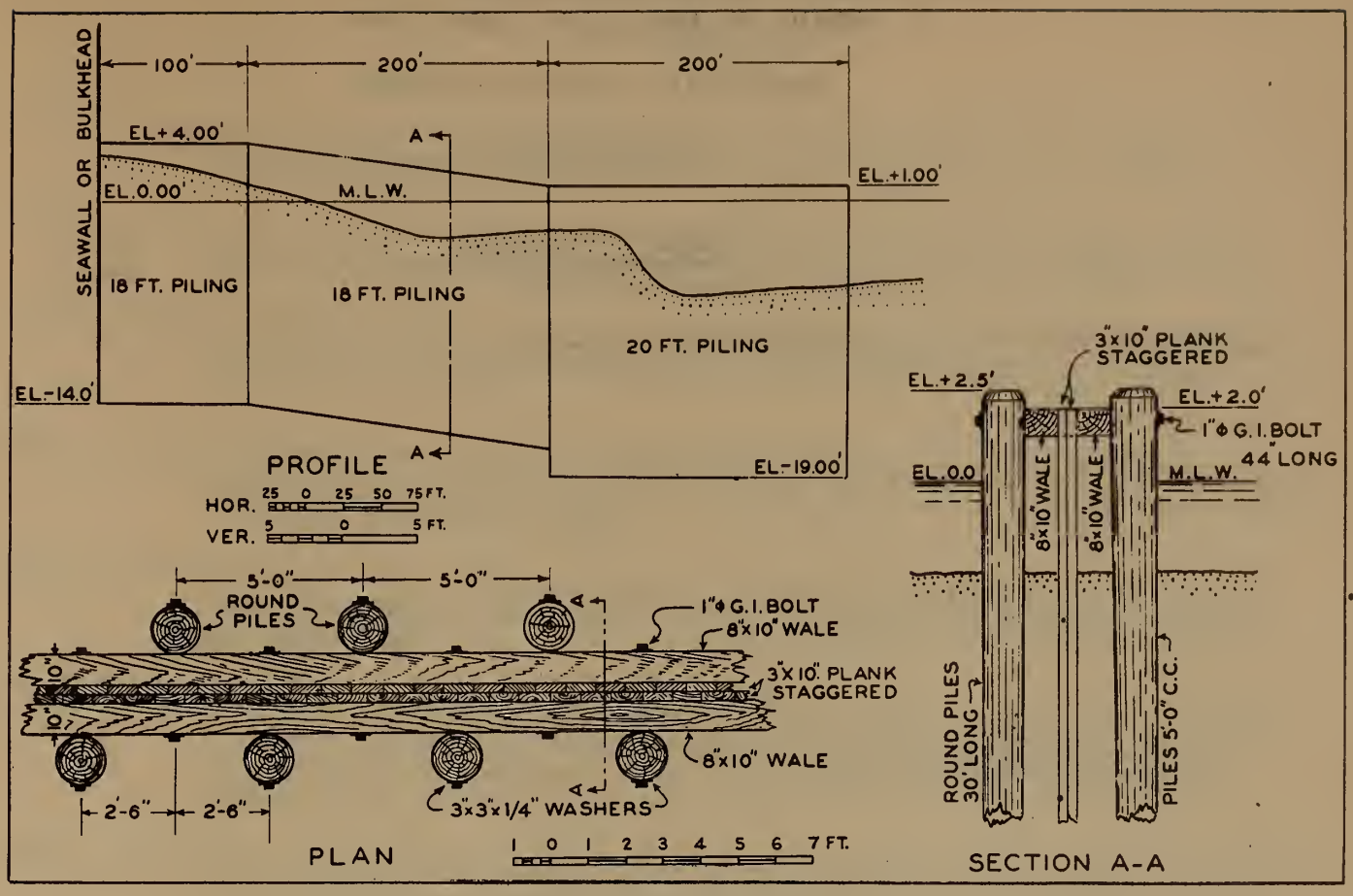



FIG. IX1-3 - Typical timber groin (41)

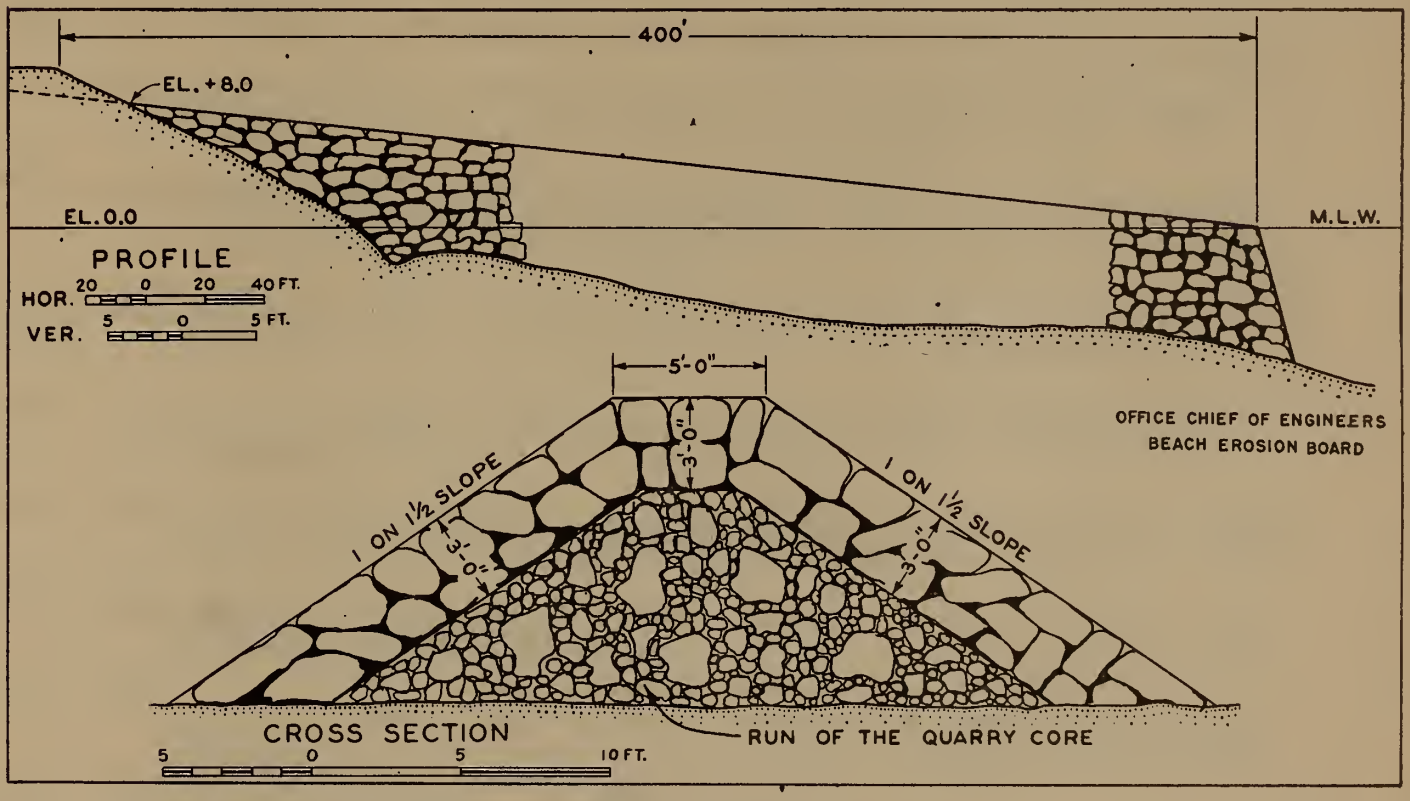



FIG. IX1-4 - Typical stone groin (41)





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## J. REFERENCES

IX-1. "Attack at Massacre Bay, Attu Island", (1943), Illustration in The Military Engineer, Vol. 35, n 213, July 1943.

IX-2. Ayers, B. K., Jr. (1950), "Stanolind Platform Also Tested in Gulf of Mexico Hurricane", The Oil and Gas Journal, Feb. 23, 1950, P. 150.

IX-3. Ayers, James R. (1950), Seawalls and Breakwaters, University of California, Institute on Coastal Engineering, University Extension, October 11-13, 1950.

IX-4. Beach Erosion Board (1940), A Model Study of the Effect of Submerged Breakwaters on Wave Action. Technical Memo, No. 1, Chief of Engineers, U. S. War Department, 1940.

IX-5. Beach Erosion Board (1944), A Method for Determining Depths over Bottom Discontinuities, Technical Memo No. 5, Chief of Engineers, U. S. War Department, 1944.

IX-6. Beckett, H. A., (1948), "Some Aspects of the Design of Flexible Bridging, Including 'Whale' Floating-Roadways", The Civil Engineer in War, Vol. 2, Docks and Harbors, The Institution of Civil Engineers, London, 1948, pp. 385-400.

IX-7. Blue, F. L. and J. W. Johnson, (1949), "Diffraction of Water Waves Passing Through a Breakwater Gap", Transactions, American Geophysical Union, Vol. 30, No. 5, October 1949, pp. 705-718.

IX-8. "The Brasher Air Breakwater", (1916), Engineer, London, Vol. 122, 1916.

IX-9. Brasher, P., (1916), "The Air Breakwater Put to Severe Test", South American, Vol. 114, n 16, April 1916, P. 403.

IX-10. Building the Navy's Bases in World War II, (1947), Vol. 2, Department of the Navy, Bureau of Yards and Docks, U. S. Government Printing Office, Washington D. C., 1947.

IX-11. "Calming Rough Water with Compressed Air", (1916), Engineering News, Vol. 75, n 25, June 22, 1916.

IX-12. Cappel, C. G., (1950), "Offshore Drilling Platforms", The Oil and Gas Journal, February 23, 1950, pp. 142-145.

IX-13. Carr, J. H., (1951), Mobile Breakwater Studies, Report No. N-64.2, Hydrodynamics Laboratories, California Institute of Technology

IX-14. Civil Engineer Corps Bulletin, Cover, (1951), U. S. Navy, Vol. 5, n 51, February 1951, Navdocks p. 2.

IX-15. Combs, L. B., (1944), "Innovations of Amphibious Warfare", The Military Engineer, Vol. 36, No. 220, February 1944, pp. 44-47.

IX-16. Concrete Shore Protection, Portland Cement Association.

IX-17. The Corps of Engineers, (1951), Photographs - San Francisco Office.

IX-18. Dean, W. R. (1945a), On the Reflection of Surface Waves by a Submerged Plane Barrier, Ministry of Supply, Wave Report No. 8, 1945.

IX-19. Dean, W. R. (1945b), On Some Cases of the Reflection of Surface Waves by an Inclined Plane Barrier, Ministry of Supply, Wave Report No. 13, 1945.

IX-20. Division Engineers, (1938), Study of Wave Forces, Prepared in Office of the Division Engineers, Great Lakes Division, Cleveland, Ohio, Oct. 1938.

IX-21. Doane, K. P., (1945), "Harbor Construction at Peleliu", The Military Engineer, Vol. 37, n 240, October 1945, pp. 414-418.

IX-22. Dubrow, M. D. and A. C. Rayner (1948), "Recent Storm Damage Along the Coasts", The Military Engineer, Vol. 40, n 269, March 1948, pp. 120-124.

IX-23. Engineering Manual For Civil Works (Part X, Structural Design), Chapter 7, "Breakwaters", Corps of Engineers, Portland District, Oregon,

IX-24. Farley, R. C. and J. S. Leonard (1950). "Gulf Hurricane", The Oil and Gas Journal, February 23, 1950, pp. 136-141.

IX-25. Fleming, R. H. and C. C. Bates, (1950), The Engineering Application of Sea and Swell Data. University of California, Institute of Coastal Engineering, University Extension, October 11-13, 1950.

IX-26. Fortson, E. P. (Jr); F. R. Brown; R. Y. Hudson; H. B. Wilson; and H. A. Bell, (Jr.), (1949), Wave and Surge Action Monterey Harbor, Model Investigation, Tech. Memo No. 2-301 Waterways Experiment Station, Vicksburg, Mississippi, September 1949.

IX-27. Gaillard, D. D. (1904), Wave Action in Relation to Engineering Structures, The Engineer School, Fort Belvoir, Va., 1904.

IX-28. Bogolepoff, I. A., (1937), "The Compressed Air Breakwater", Bulletin, Permanent International Association of Navigation Congresses, Vol. 12, No. 23, January 1937.

IX-29. Gorlinski, J. S., (1944), "Naples: Case History in Invasion", The Military Engineer, Vol. 36, n 222, April 1944, pp. 109-114.

IX-30. Hall, W. C., (1942), "Beach Protection Measures", The Military Engineer, Vol. 34, n 200, June 1942, pp. 292-296.

IX-31. Hamilton, W. S. (1950), "Forces Exerted by Waves on a Sloping Board", Transactions, American Geophysical Union, Vol. 31, No. 6, 1950, pp. 849-855.

IX-32. Hammond, D. G., (1944), "Aleutian Water Transportation Problems", The Military Engineer, Vol. 36, n 229, November 1944, pp. 373-375.

IX-33. Hannum, W. T., (1942), "Construction in the Islands of the Pacific", The Military Engineer, Vol. 34, n 196, February 1942, pp. 63-68.

IX-34. Heaney, W. F., (1944), "Amphibious Engineers in Action", The Military Engineer, Vol. 36, n 223, May 1944, pp. 145-152.

IX-35. Heaney, W. F., (1947), "Down Ramp", Chapter XIII, The Military Engineer, Vol. 39, n 255, January 1947.

IX-36. Heins, A. E. (1950), "Water Waves Over a Channel of Finite Depth with a Submerged Plane Barrier", Canadian Journal Mathematics II, No. 2, 1950, pp. 210-222.

IX-37. Henney, F. A., (1944), "Combat Engineers in North Africa, Part I, The Capture of Port Lyantey", The Military Engineer, Vol. 36, n 219, January 1944, pp. 1-4.

IX-38. Hickson, R. E. and F. W. Rodolf, (1950a), Design and Construction of Jetties, University of California, Institute on Coastal Engineering, University Extension, October 11-13, 1950.

IX-39. Hickson, R. E. and F. W. Rodolf, (1950b), Case History of Columbia River Jetties, University of California, Institute on Coastal Engineering, University Extension, October 11-13, 1950.

IX-40. Hiltabidle, W. O., (1945), "Pontoon Piers on Guam", The Military Engineer, Vol. 37, n 235, May 1945, pp. 180-184.

IX-41. Horton, D. F. (1950), The Design and Construction of Groins, University of California, Institute on Coastal Engineering, University Extension, October 11-13, 1950.

IX-42. Hudson, R. Y., (1945), "Model Tests of Portable Breakwaters for D-Day Invasion Harbors", Civil Engineering, Vol. 15, n 9, September 1945, p. 405.

IX-43. Hudson, Robert Y. and L. Frank Moore, (1950), The Hydraulic Model as an Aid in Breakwater Design, Institute on Coastal Engineering, University of California, Institute on Coastal Engineering, University Extension, October 11-13, 1950.

IX-44. Hurley, W. M., (1944), "The Flow of Engineer Supplies", The Military Engineer, Vol. 36, n 226, August 1944, pp. 254-255.

IX-45. Jefferys, H., (1944), Note on the Offshore Bar Problem and Reflection from a Bar, Ministry of Supply, Wave Report No. 3, June 1944.

IX-46. Jellett, J. H. (1948), "The Lay-out, Assembly, and Behaviour of the Breakwaters Arromanches Harbour (Mulberry 'B')", The Civil Engineer in War, Vol. 2, Docks and Harbors, The Institution of Engineers, London, 1948, pp. 291-312.

IX-47. John, F. (1948), "Waves in the Presence of an Inclined Barrier", Quarterly, The Institute for Mathematics and Mechanics, New York University, Vol. 1, No. 2, May 1948, pp. 149-200.

IX-48. Johnson, J. W., R. A. Fuchs and J. R. Morison, (1951), The Damping Action of Submerged Breakwaters, University of California, Institute of Engineering Research, Technical Report Series 3 Issue 322, April 1951.

IX-49. Kaplan, K. and H. E. Pope, Jr. (1950), Design of Breakwaters, University of California, University Extension, Institute of Coastal Engineering, October 11-13, 1950.

IX-50. Knight, C. W. and S. G. Stock, (1949), "Nos. 1 and 2 Military Ports", The Civil Engineer in War, Vol. 2, Docks and Harbors, The Institution of Civil Engineers, London, 1948, pp. 3-35.

IX-51. Lamb, H. (1945), Hydrodynamics, 6th Edition, 1945, p. 262.

IX-52. Lochner, R., O. Faber and W. G. Penney (1948), "The 'Bombardon' Floating Breakwater", The Civil Engineer in War, Vol. 2, Docks and Harbors, The Institution of Civil Engineers, London, 1948, pp. 256-290.

IX-53. Manning, J. J. "Normandy's Artificial Harbors", The Military Engineer, Vol. 36, n 230, December 1944, pp. 389-394.

IX-54. McOuat, H. W., (1950), Case History of Los Angeles Harbor, University of California, Institute on Coastal Engineering, October 11-13, 1950.

IX-55. Middleton, J. K. (1948). "Repairs to Mediterranean Ports, 1942-1945", The Civil Engineer in War, Vol. 2, Docks and Harbors, The Institution of Civil Engineers, London, 1948, pp. 148-172.

IX-56. The Military Engineer, (1948), Cover, Vol. 40, n 271, May 1948.

IX-57. The Military Engineer, (1951), Illustration, Vol. 43, n 291, Jan-Feb. 1951.

IX-58. Minikin, R. R., (1950), Winds, Waves, and Maritime Structures, Charles Griffin and Co. Ltd., London, 1950.

IX-59. Moore, C. R., (1945), "Engineer Supply in the Continental Invasion", The Military Engineer, Vol. 37, n 242, December 1945, pp. 475-484.

IX-60. Morison, J. R. (1949a), Model Study of Wave Action on Underwater Barriers, University of California, Institute of Engineering Research, Berkeley, Report No. HE-116-304, 1949.

IX-61. Morison, J. R. (1949b), "Measurement of Heights by Resistance Elements", The Bulletin, Beach Erosion Board, Chief of Engineers, Vol. 3, No. 3, 1949, pp. 16-22.

IX-62. Munk, W. H., (1948), "Wave Action on Structures", AIME, Technical Publication No. 2322, March 1948.

IX-63. Noyes, J. R., (1939), "Land Reclamation and Shore Protection in the Netherlands", The Military Engineer, Vol. 31, n 177, May-June 1939, pp. 212-216.

IX-64. O'Brien, M. P. (1949), "The Causes of Plunging and Spilling Breakers", The Bulletin, Beach Erosion Board, Chief of Engineers, Vol. 3, No. 3, 1949, pp. 7-10.

IX-65. Patterson, C. B. and W. B. Tanner (1950), Review of Model Study for Breakwater Location, Alameda Naval Air Station, Waterways Experiment Station, Vicksburg, Mississippi, March 1950.

IX-66. Pavry, R. (1948a), "The Composite Pier", The Civil Engineer in War, Vol. 2, Docks and Harbors, The Institution of Civil Engineers, London, 1948, pp. 141-147.

IX-67. Pavry, R. (1948b), "Mulberry Pierheads", The Civil Engineer in War, Vol. 2, Docks and Harbors, The Institution of Civil Engineers, London, 1948, pp. 369-384.

IX-68. Peel, Kenneth P., (1950), Location of Harbors, University of California, Institute on Coastal Engineering, University Extension, October 11-13, 1950.

IX-69. Pike, J. E., (1950), "An Insurance Counselor's Views on Offshore Risks", The Oil and Gas Journal, February 23, 1950, pp. 146-149.

IX-70. "Recent Landing Operation of American Troops", (1943), Illustration in The Military Engineer, Vol. 35, n 214, August 1943.

IX-71. Schaufele, H. J., (1950), Erosion and Corrosion on Marine Structures, University of California, Institute on Coastal Engineering, University Extension, October 11-13, 1950.

IX-72. "The Seabees in the Normandy Invasion", Engineering News-Record, Dec. 28, 1944, p. 54.

IX-73. Seiwell, H. R., (1947), "Military Oceanography in World War II", The Military Engineer, Vol. 39, n 256, May 1947, pp. 202-210.

IX-74. Skerrett, R. G., (1921), "Smashing Angry Seas with Bubbles of Compressed Air", Compressed Air Magazine, Vol. 26, n 1, January 1921.

IX-75. Smith, W. H., (1945), "Artificial Harbors for Normandy Beaches", Civil Engineering, Vol. 15, n 6, January 1945, p. 255.

IX-76. Soper, G. A., (1940), "Beach Erosion Studies (A Discussion)", Transactions, American Society of Civil Engineers, Vol. 105, 1940, pp. 893-895.

IX-77. Stocking, H. E. (1944), "Coral Reefs of the South Pacific", The Military Engineer, Vol. 36, n 226, August, 1944, pp. 249-253.

IX-78. Streblow, A. G., (1950), Breakwater Construction, University of California, Institute on Coastal Engineering, University Extension, October 11-13, 1950.

IX-79. Stucky, A. and D. Bonnard, (1937), "Contribution to the Experimental Study of Marine Rock Fill Dikes", Bulletin Technique de la Suizze Romande, August, 1937.

IX-80. Todd, F. H., (1948), "Model-Experiments on Different Designs of Breakwaters", The Civil Engineer In War, Vol. 2, Docks and Harbors, The Institution of Civil Engineers, London, 1948, pp. 243-255.

IX-81. Tratman, E. E. R., (1940), "Submerged Barriers for Shore Protection", The Engineer, London, Vol. 169, March 1940, p. 226.

IX-82. Ursell, F., (1947), "The Effect of a Fixed Vertical Barrier on Surface Waves in Deep Water", Proc. Cambridge Philosophical Society, Vol. 43, 1947, pp. 374-382.

IX-83. Ursell, F., (1950), "Surface Waves in Deep Water in the Presence of a Submerged Circular Cylinder", Proc. Cambridge Philosophical Society, Vol. 46, 1950, pp. 141-158.

IX-84. Wakefield, W. A., (1948), "Bailey-Type Piers and Wharfs", The Civil Engineer in War, Vol. 2, Docks and Harbors, The Institution of Civil Engineers, London, 1948, pp. 112-118.

IX-85. Wakefield, W. A. and E. J. Crabb, (1948), "Preparation of R. E. (Transportation) Units for Emergency Port Repairs", The Civil Engineer in War, Vol. 2, Docks and Harbors, The Institution of Civil Engineers, London, 1948, pp. 66-73.

IX-86. Wiegel, R. L. (1948), "Diagrams and Tables of Relationships Commonly used in Investigations of Surface Waves", The Bulletin of the Beach Erosion Board, Special Issue No. 1, July 1948.

IX-87. Wilson, W. S. and F. W. Sully, (1948), "The Traversing and Side-Launching of Heavy Craft", The Civil Engineer in War, Vol. 2, Docks and Harbors, The Institution of Civil Engineers, London, 1948, pp. 211-228.

IX-88. Winn, H. P., (1944), "The Military Engineer and Supply", The Military Engineer, Vol. 36, n 226, August 1944, pp. 256-259.

IX-89. Wood, C. R. J., (1948a), "Phoenix", The Civil Engineer in War, Vol. 2, Docks and Harbors, The Institution of Civil Engineers, London, 1948, pp. 336-368.

IX-90. Wood, C. R. J., (1948b), "Reinforced-Concrete Pier Pontoons and Intermediate Pierheads Pontoons", The Civil Engineer in War, Vol. 2, Docks and Harbors, The Institution of Civil Engineers, London, 1948, pp. 40-425.

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